CONTROL STRATEGY FOR SIGNALIZED INTERSECTIONS

FINAL REPORT

Research Project ~~130~~ 132, FC# 97-35

Prepared for

Idaho Transportation Department
Traffic Division

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March 1998

NCATT

Report Number 98-14-01

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1.0 EXECUTIVE SUMMARY

This report documents the findings of a corridor study conducted by the National Center for Advanced Transportation Technology (NCATT) at the University of Idaho. The transportation corridor analyzed in this report involved twelve signalized intersections on the US-95 north corridor. The existing traffic control system consists of two closed loop fully actuated control systems. The southern system has four signalized intersections, and the northern system has eight signalized intersections.

1.1 Purpose and Objective

The purpose of this study is to determine existing traffic condition and recommend a signal-timing plan that will provide a smooth progression and maintain a level of service (LOS) "D" at each intersection throughout the entire corridor. To achieve this objective, NCATT and ITD performed the following field reviews to establish existing traffic conditions on US-95.

- Collection of traffic counts at the intersections of US-95 with Ironwood, Eastbound Ramp, Westbound Ramp, Appleway, Neider, Bosanko, Kathleen, Dalton, Hanley, Canfield, Prairie, and Hayden.
- Collection of signal phasing and timing at each intersection mentioned above.
- Collection of speed and delay with ITD test vehicle.
- Collection of saturation headway at two critical intersections, Kathleen and Appleway with four video cameras.
- Accident review from 1990 to 1996.
- Origin-destination study.

1.2 Potential Coordination difficulties

The potential difficulties in coordinating these traffic signals are identified as follows:

- *Speed limit variation* - speed limit varies from 35 to 45 mph.
- *Mid-block volumes* - land use and economic activity along US-95 generate significant mid-block volumes.
- *Pedestrian crossing* - long pedestrian time is required due to the width of US-95, some intersections have pedestrian crossing lengths of 140 feet.

- *Spacing between adjacent intersections* - intersection spacing varies from as little as 440 feet near the I-90 Interchange to over a mile long between Prairie and Hayden intersections.

1.3 Existing Traffic Condition

An analysis of existing traffic condition of the corridor was performed. The following is a summary from the evaluation:

- HCS analysis shown in Table 3 indicated that all intersection performed at LOS "F", except Dalton, Neider, and Eastbound ramp performed at LOS "D" and LOS "C" for Kathleen. Further analysis indicated that poor progression and signal settings cause the excessive delay.
- The saturation headway study indicated average of 2.08 seconds for through movement and 2.22 seconds for left turn movement.
- An Origin-Destination (O-D) study showed that most motorists travel between Ironwood and Appleway. Sub-systems were then analyzed, but no major benefits were found. Therefore, the signal timing strategy remained as one continuous system between Ironwood and Hayden.

1.4 Signal Timing Settings

The current study focused on four signal-timing plans:

- EXISTING - the original signal timing plan that was under operation during data collection back in October 1996. The signal control was fully actuated.
- APPLIED - the 140 seconds background cycle length signal-timing plan implemented by Idaho Transportation Department personnel in Fall 1997.
- PROPOSED - the 115 seconds background cycle length signal timing that resulted from data analysis, optimization, and evaluation by the National Center for Advanced Transportation Technology (NCATT) research team at the University of Idaho.
- MIXED – combined the signal timings of APPLIED and PROPOSED. PROPOSED signal timing is applied from Ironwood to Appleway and APPLIED signal timing is used in the remaining eight intersections from Neider to Hayden.

1.5 Traffic Control Strategies

The strategies of determine proper signal timing including the use of the following computer models:

- HCS – evaluate isolated intersections.
- SIDRA – evaluate and optimize cycle length and signal phases and timing for isolated intersection.
- SIGNAL94 / TEAPAC – evaluate and optimize cycle length and signal phases and timing for isolated intersection.
- PREPASSER / TEAPAC and PASSER II-90 – optimize progression bandwidth of arterial.
- PRETRANSYT / TEAPAC and TRANSYT-7f – minimize delays and stops for arterial or network.
- PRENETSIM / TEAPAC and TRAF-NETSIM – simulate corridor traffic flow at microscopic level.
- CORSIM- an updated version of TRAF-NETSIM and FRESIM.

1.6 Measure of Effectiveness

Measures of effectiveness (MOEs) from CORSIM output, including speed, delay, and queue lengths for the major through movements, were compared to evaluate the signal timings for both fall 1996 and summer 1997 traffic volumes shown in Appendix I-VI and I-VII. The US-95 arterial MOEs for EXISTING, PROPOSED, and APPLIED signal timings are compared and discussed in section 5.3. Appendix I-VIII shows an overall US-95 arterial comparison between PROPOSED vs. EXISTING and APPLIED vs. EXISTING. Table 1 is the summary of Appendix I-VIII on comparison between PROPOSED vs. EXISTING and APPLIED vs. EXISTING. Positive percentages indicate improvements over EXISTING signal timing, whereas negative percentages indicate decreases in performance.

Table 1. Arterial MOEs for Fall 1996 Traffic Volume

Arterial Through Movement	PROPOSED vs. EXISTING Percent Improved		APPLIED vs. EXISTING Percent Improved	
	NB	SB	NB	SB
Speed	15.86%	0.00%	-7.16%	-7.35%
Delay	52.89%	33.08%	-33.70%	-10.35%
Queue Length	91.34%	33.29%	-39.70%	-5.98%

PROPOSED signal timing was evaluated under summer volume to ensure that it met summer's high tourist season conditions. Since the APPLIED signal-timing plan is currently in operation, the PROPOSED signal-timing plan is compared with the APPLIED signal timings for summer volume. The results are shown in Table 2.

Table 2. Arterial MOEs for Summer 1997 Traffic Volume

Arterial Through Movement	PROPOSED vs. APPLIED Percent Improved	
	NB	SB
Speed	18.45%	3.65%
Delay	53.06%	37.12%
Queue Length	87.93%	39.55%

The APPLIED signal-timing plan has significantly improved the progression along the US-95 through movements over EXISTING signal timing. The APPLIED signal-timing plan used 140 seconds background cycle length to produce an impressive progression bandwidth of over 40 seconds. Generally, long cycle lengths contribute long delays on the overall intersection performance, especially for the minor movements. Thus, careful selection of background cycle length, offsets, and splits are crucial elements in minimizing delays along arterials and minor streets. A proper signal-timing plan can improve the overall traffic operation with reduced travel time, decreased delay, and a system wide reduction in fuel consumption and air pollution.

The PROPOSED signal-timing plan was developed based on a system wide performance strategy that would minimize delays to all major and minor traffic movements to an acceptable LOS "D" or better. Table 3 shows the LOS for all intersections. Progression bandwidth along US-95 is also considered as shown in Table 4. Progression bandwidth is determined based on directional volume. A longer bandwidth is given for southbound (SB) direction during morning traffic hours because motorists travel to Central Business District (CBD) for work, whereas, a longer bandwidth is provided to serve northbound (NB) afternoon traffic for motorists leaving the CBD.

The MIXED signal-timing plan was evaluated but no apparent improvement over APPLIED nor PROPOSED signal-timing plans shown in Appendix I-VI and I-VII.

Table 3. HCS LOS for EXISTING, APPLIED and PROPOSED signal timings

Intersection	EXISTING LOS	APPLIED LOS	PROPOSED LOS
Hayden	F	F	D
Prairie	F	D	B
Canfield	F	C	C
Hanley	F	F	C
Dalton	D	F	B
Kathleen	C	C	C
Bosanko	~	C	B
Neider	D	D	C
Appleway	F	~	D
WB Ramp	F	~	B
EB Ramp	D	~	C
Ironwood	F	~	D

Note: "~" indicates field data not available

Table 4. Progression Bandwidth of PORPOSED Signal Timing

	AM		MID-DAY		PM	
	NB	SB	NB	SB	NB	SB
Bandwidth (sec)	14	43	26	21	29	21

1.7 Conclusion and Recommendation

The research project reached the following conclusions:

- Based on the collected field traffic volumes, no major capacity deficiencies were found for any the intersection within the arterial. Queuing problems may be due to poor progression.
- The analysis of sub-systems showed that no major benefits were obtained by dividing the system into sub-systems. Although it may be difficult to maintain progression for the intersections at Hayden and Prairie due to the long distance between them, they are included in the system to at least progress those possibly well maintained platoons.
- A cycle length of 115 seconds was found to be the best for all the time periods. Some intersections require phasing modification from the existing phasing. These intersections include Dalton, Kathleen, Appleway, WB ramp, and Ironwood.
- The use of CORSIM was valuable in evaluating timing plans before implementation.

Based on evaluation of all three-signal setting plans in the CORSIM simulation model. NCATT research team recommends the PROPOSED signal timing plan. This signal timing plan provided an acceptable LOS “D” or better for both major and minor movements, and a smooth progression along US-95 priority stream.

2.0 ABSTRACT

Corridor traffic signal-timing synchronization is one of the most cost-effective methods for reducing delays and improving the overall operation along a congested corridor for all vehicles. A section of signalized traffic intersections on US-95 in northern Idaho connecting Coeur d'Alene to Hayden has generated complaints by the local motorists regarding long delays at the intersections. Traffic congestion due to the rapid population growth of Coeur d'Alene and long queuing times at critical intersections due to large number of visitors during the summer months are at the heart of these complaints. In order to provide smooth progression and fewer delays along the US-95 and its cross streets, TRANSYT-7F, PASSER II-90, TEAPAC, and CORSIM models were used to study and re-coordinate the signal-timing of the existing twelve coordinated fully actuated controlled intersections.

The research project utilized PASSER II-90 and TRANSYT-7F to optimize progression and minimize delays, respectively, for motorists at all intersections. The PRENETSIM/TEAPAC was then used to create a preliminary input file for the CORSIM simulation model. The preliminary input file was further calibrated to reflect the field data. The simulation output of the validated CORSIM model produced many measurements of effectiveness (MOE). MOEs such as speed, time delays, and queue length were compared among the EXISTING, APPLIED and PROPOSED signal-timing plans. The PROPOSED signal-timing plan showed significant improvements along the studied corridor.

3.0 INTRODUCTION

The project involved three main steps: 1) field data collection, 2) field data analysis, and 3) computer model analysis and evaluation. The project began with field data collection conducted by the Idaho Transportation Department and the University of Idaho. The data included peak hour traffic volumes, existing signal timings, intersection geometric configuration, travel speed, and saturation flow rates at key locations were collected on October 24-25, 1996. In addition, ITD and NCATT collected origin-destination data during the summer 1997.

The field data was then analyzed to determine the following key parameters for further study:

- Peak hours and peak 15-minute for the three periods morning, mid-day, and afternoon.
- Peak Hour Factors.
- Mid-block volumes.
- Saturation headways.
- Most common cycle extracted from field signal timings.

The third step involved the utilization of computer models to analyze, design, and evaluate the performance of each individual intersection as well as the entire arterial system. The Highway Capacity Software (HCS) and SIGNAL 94/TEAPAC were used to assess LOS for each isolated intersection. PASSER II-90 and TRANSYT 7-F were used to coordinate the US-95 corridor. Finally, the PROPOSED signal-timing plan was evaluated under microscopic traffic simulator CORSIM for its measure of effectiveness.

4.0 FIELD DATA

Field data can be classified into five main groups as follows:

- Geometric data
- Traffic volume data
- Signal timing data
- Accident data
- Origin-destination data

4.1 GEOMETRIC CONFIGURATION

The study area involved twelve signalized intersections on the US-95 north corridor shown in Figure 1. The existing traffic system consists of two closed-loop systems. The corridor extends from Coeur d'Alene, Idaho to Hayden, Idaho. The twelve consecutive signalized intersections heading northbound are Ironwood, East ramp, West ramp, Appleway, Neider, Bosanko, Kathleen, Dalton, Hanley, Canfield, Prairie, and Hayden shown in Figure 2. The segment of US-95 from Neider to Hayden is a four-lane divided highway classified as major arterial with a wide median. The south segment extending from Ironwood to Appleway is a four-lane arterial but has no medians. Northbound and southbound approaches at all intersections have two through lanes and exclusive left and right turning lanes. Except at Appleway, which has three through lanes in the northbound direction. The cross street approaches vary among intersections. A schematic layout showing the number of lanes and their widths on each approach of all the intersections is shown in Appendix A.

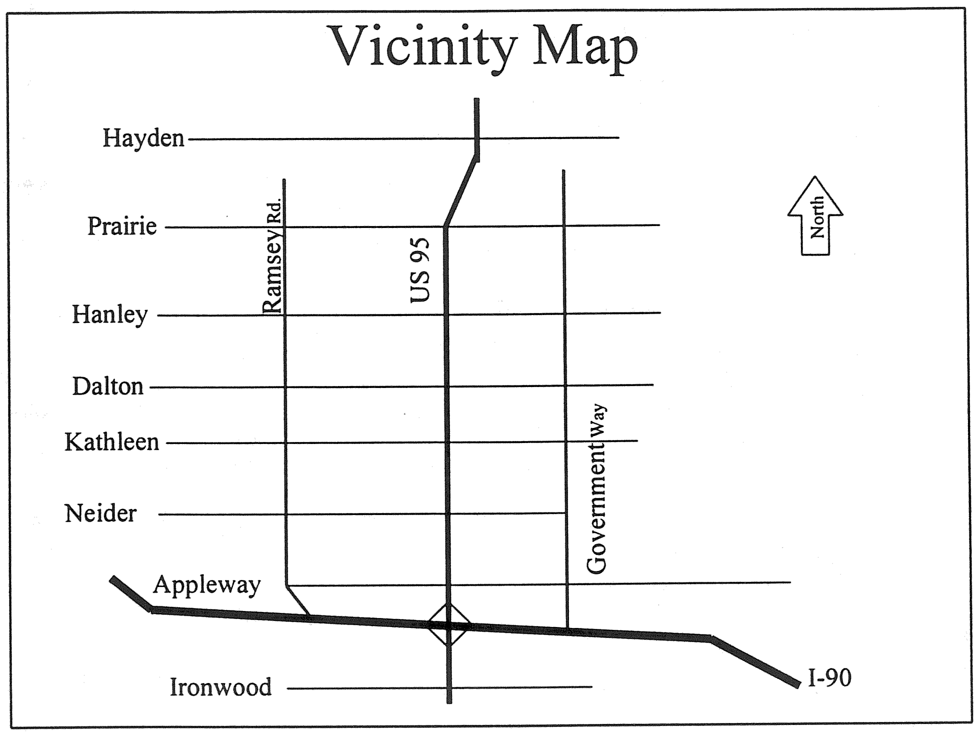


Figure 1. Vicinity Area Map

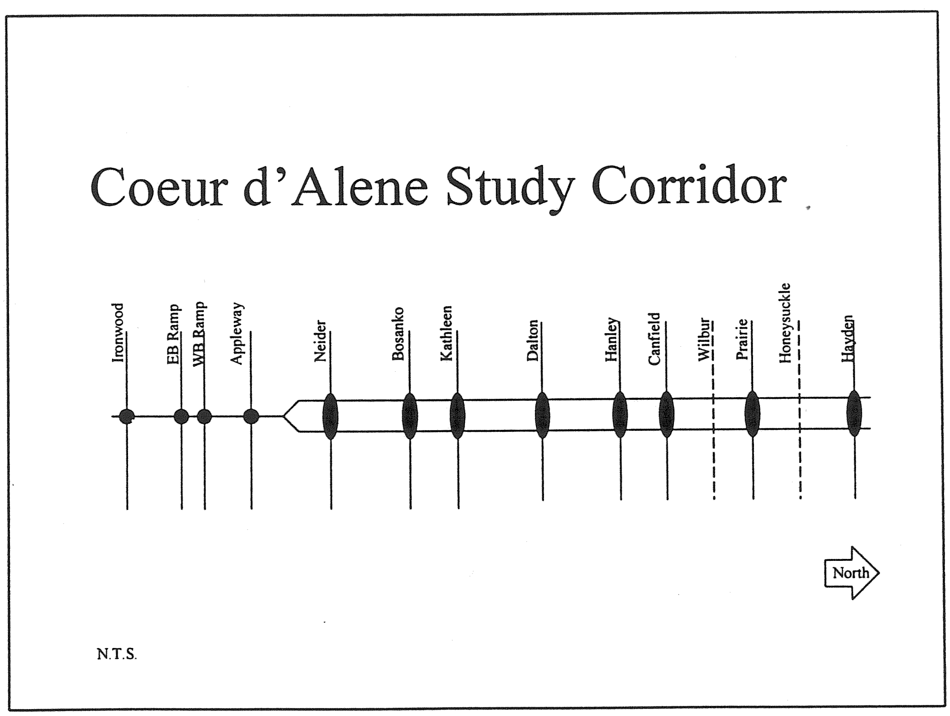


Figure 2. Study Corridor

4.2 TRAFFIC DATA

4.2.1 Vehicle Volume Collection

The field data collection included the traffic turning movements, heavy vehicle volume, traffic signal controller timings, and test vehicle speed and delay for the analysis periods of AM, mid-day, and PM.

- AM period - 6:30 a.m. to 9:30 a.m.
- Mid-day period - 11:00 a.m. to 1:00 p.m.
- PM period - 3:30 p.m. to 6:30 p.m.

A group of over 40 people from the Idaho Transportation Department and the University of Idaho participated in the data collection. Video cameras were set up to measure the saturation headway at two critical intersections, Kathleen and Appleway. Traffic volumes were manually collected for each approach of eleven intersections during the three analysis periods for two days, Thursday and Friday (Bosanko was under construction at the time of data collection). The volume data was entered in vehicle volume summary sheets. Then traffic volume data were analyzed to determine the peak hours and the peak 15-minutes so system wide analysis could be performed. The most critical period was the Friday afternoon peak 15-minute traffic volume. Hence, it was defined as the critical analysis period.

4.2.2 Traffic Flow Profile

Traffic flow profile in terms of arrival and departure flow rates demonstrates the arterial flow patterns. The arrival and departure approach movements for each individual intersection are illustrated in Figure 3. For the northbound direction, arrival flow is defined as the sum of the northbound left turn, northbound through, and northbound right turn volumes, and the departure flow is equal to the sum of eastbound left turn, northbound through, and westbound right turn volumes. The same definition was applied to the southbound direction.

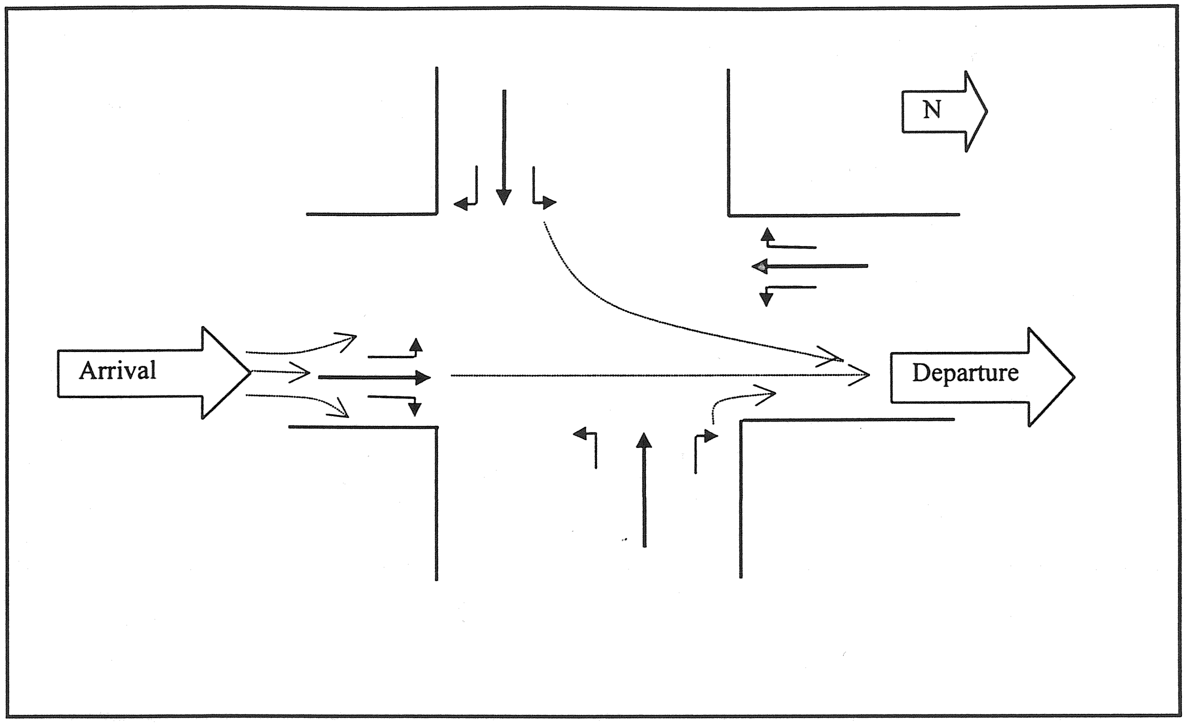


Figure 3. Approach Movements

The flow patterns for the Friday peak 15-minute (5:00 - 5:15 p.m.) were analyzed. The arrival and departure flow patterns for both northbound and southbound directions are shown in Figure 4. From the plots, it is observed that the northbound direction has a high flow rate at Appleway Street, whereas, for the southbound direction, Neider has high values for both arrival and departure flows. But no major capacity deficiencies were found for any intersection within the arterial.

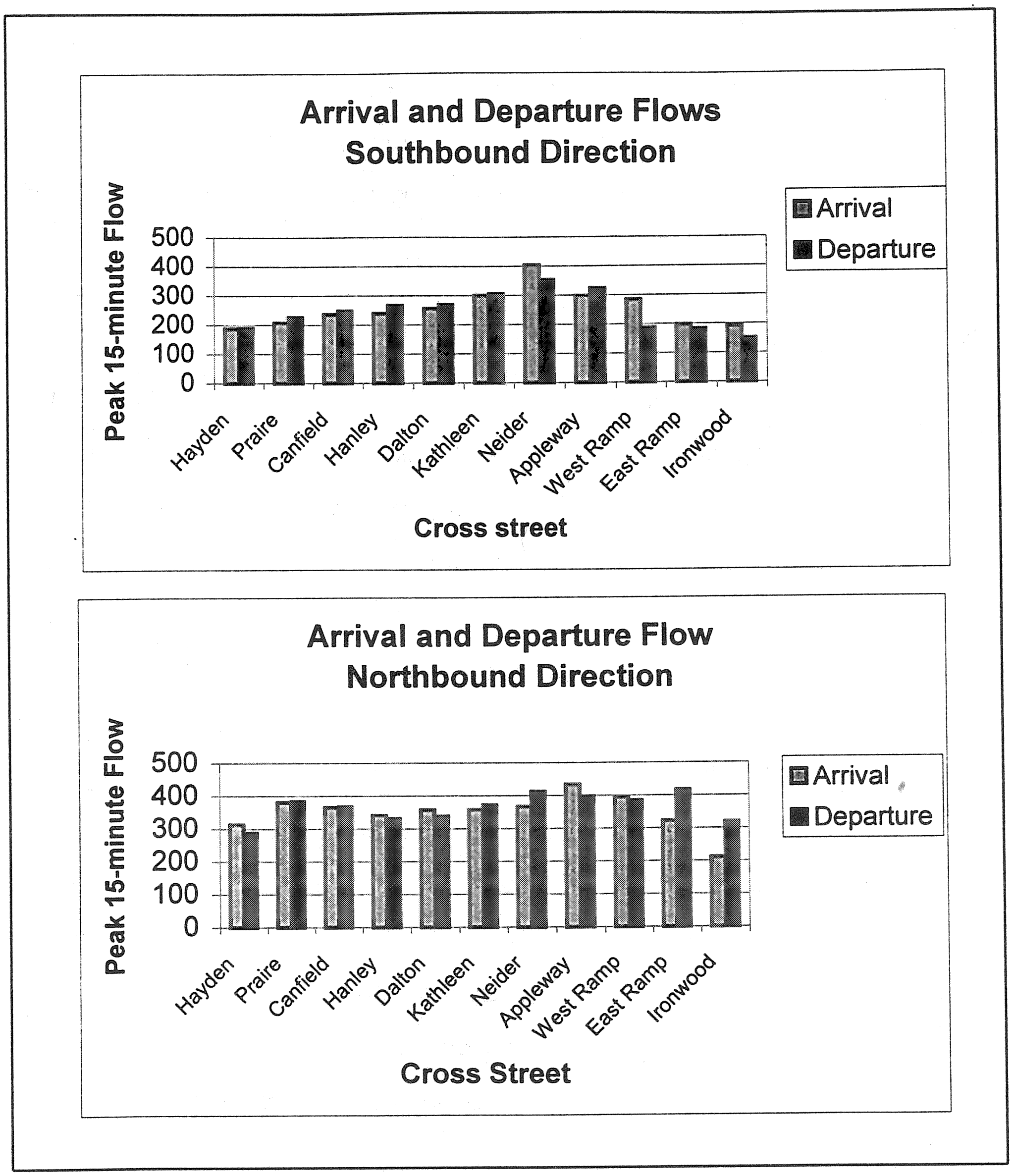


Figure 4. Traffic Flow Profile

4.2.3 Heavy Vehicle Volume

Heavy vehicles volume was observed for three peak periods at Hanley cross street. The averages heavy vehicle percentages in the 5:00 - 5:15 time interval was calculated and used for the computer model study. Two percent of heavy vehicles were used for both left and right turns and six percent were used for through movement.

4.2.4 Saturation Headway

Saturation headway is defined as the average time headway of all vehicles starting from the fourth vehicle of each cycle. The time headway is defined as the elapsed time between the passage of identical points on two consecutive vehicles. The saturation headway was conducted using four video cameras at two critical intersections, Kathleen and Appleway. Northbound and westbound traffic were recorded at Appleway, and southbound and eastbound traffic were recorded for Kathleen. The recorded tapes were played back and time headways were observed using a stopwatch. The front bumper was used as a reference point. The headways were determined for all lanes of the recorded approaches. For every cycle, saturation headway for each lane was calculated. A summary of the saturation headways of all the lanes for the observed approaches for these two intersections is shown in Table 5. Saturation flow rate is determined based on the reciprocal value of saturation headways. The saturation flow rates for left and through movements are 1625 vph and 1733 vph, respectively. No sufficient right turn headway data were observed so the saturation flow rate of 1600 vph is assumed for the right turn movement.

Table 5. Saturation Headways for the peak 15 minute period

Tape #	Cross Street	Direction	Period	Saturation Headway (seconds) Left	Through
3	Appleway	Northbound	Mid-day	2.2	1.96
					2.18
					2.17
4	Appleway	Northbound	Afternoon	1.46	1.87
					1.84
					2.12
6	Appleway	Eastbound	Afternoon	2.11	2.38
					2.03
9	Appleway	Eastbound	MID-DAY	1.96	2.12
					1.76
10	Kathleen	Westbound	Afternoon	2.47	2.25
11	Kathleen	Southbound	Afternoon	2.37	2.36
					2.05
13	Kathleen	Southbound	Mid-day	2.94	2.13
					1.94
Overall Average Saturation Headway				2.22	2.08
Overall Average Saturation flow rate				1625	1733

4.2.5 Speed and Delay

An Idaho Transportation Department test vehicle was used to determine the speed between adjacent intersections and the delay experienced at each intersection. Using a stopwatch, arrival time and departure time at each intersection were recorded. The delay was calculated based on the difference between the arrival and departure times at each intersection. Distances between the intersections were measured from center to center of each intersection. The speed was calculated using the distances between the consecutive intersections divided by the time difference between the arrival and departure times. This speed includes the control delay of traffic devices. More than 15 speed and delay runs were performed each day so the result can be used to calibrate the CORSIM model. Average delay in seconds for the peak 15-minutes was tabulated, then plots were drawn between the cross street and its delay. For northbound and southbound directions, Appleway experiences a high delay of 62 seconds. Figure 5, 6 and 7 show the speed (with delay), running speed (without delay), and delay, respectively at each intersection for northbound and southbound directions.

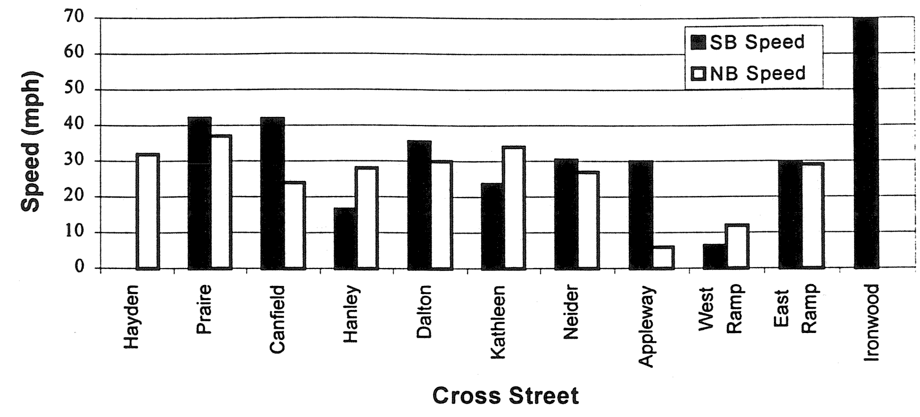


Figure 5. Speed Profile

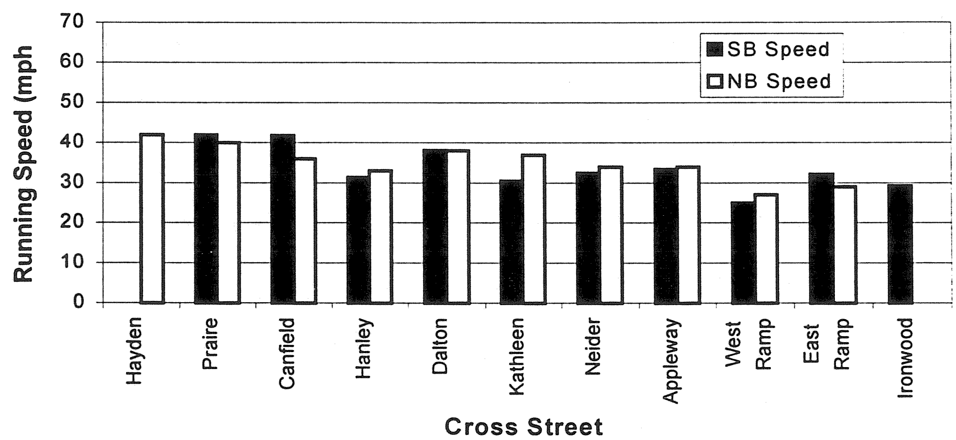


Figure 6. Running Speed Profile

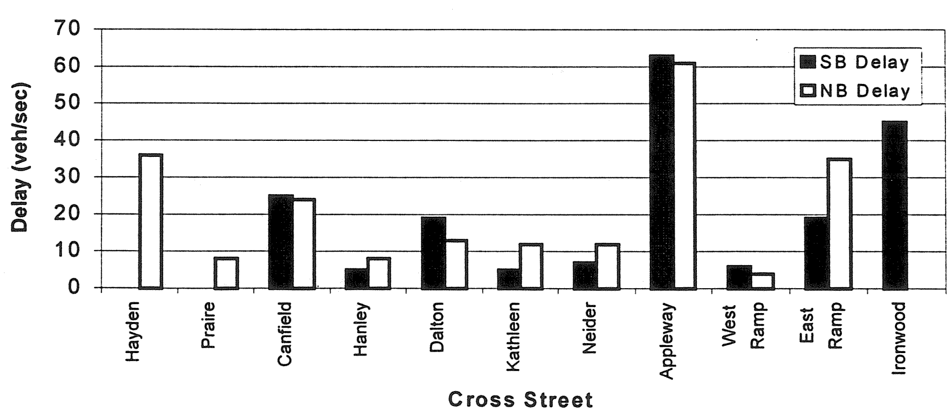


Figure 7. Intersection Delay

4.2.6 Peak Hour Factor

Peak hour factors (PHF) are calculated for all intersection of all studied periods. Peak hour factor is calculated based on the following equation (HCM, 1994):

$$PHF = \frac{V}{(4 * V_{15})}$$

Where,

V = hourly volume (vph)

V_{15}= volume during the peak 15-min of the peak hour (veh/15 min).

4.2.7 Traffic Data Summary

Summary of the traffic data is presented in Appendix B. This includes the volumes for all approaches at each intersection, total intersection volume, peak hour for each period, peak 15-minute interval of the peak hour and PHF. This overall summary is based on the field data collected on October 24-25, 1996.

4.2.8 Mid-Block Volume

Mid-block volume for Friday afternoon peak period was obtained using the following formula or refers to Figure 8 (see Appendix C).

Mid-block Volume = Downstream Arrival Flow Rate – Upstream Departure Flow Rate

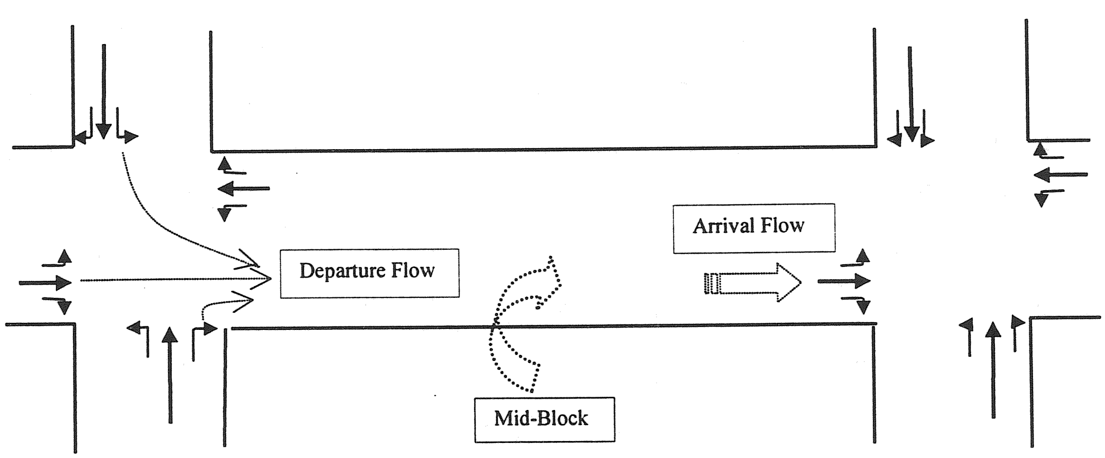


Figure 8. Mid-Block Volume

4.3 SIGNAL TIMING DATA

4.3.1 Signal Phasing Diagram

A standard eight-phase diagram used by ITD is shown in Figure 9.

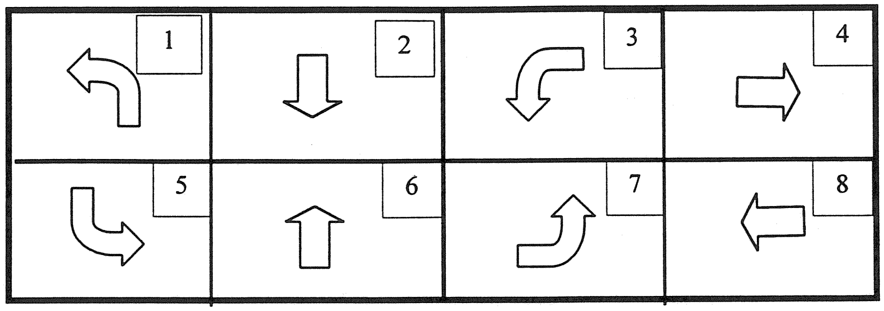


Figure 9. Phasing Diagram

4.3.2 Most Frequently Occurring Phasing Sequence

The most frequently occurring-phasing sequence and its duration were observed from the intersection signal controllers. Appendix D summarizes the most frequently occurring phasing sequences. However, the program signal timing in the traffic controller was extracted from the LM System software summarized in Appendix E. This data is used in all computer models.

4.3.3 Offsets

Offset is the difference between the green initiation times of two adjacent intersections shown in Figure 10. Offsets were calculated by obtaining the difference between the two green initiation times of the consecutive intersections based on most frequently occurring signal timing. The offsets calculated from LM system software were all positive. Offsets were calculated for consecutive intersections for both north and southbound directions. If the offset value was above 120 seconds, then 120 was subtracted from the value to assure the value is within cycle length. This is because it is assumed that the offset value should be between 0 and the cycle length. Appendix F illustrates the offsets determination.

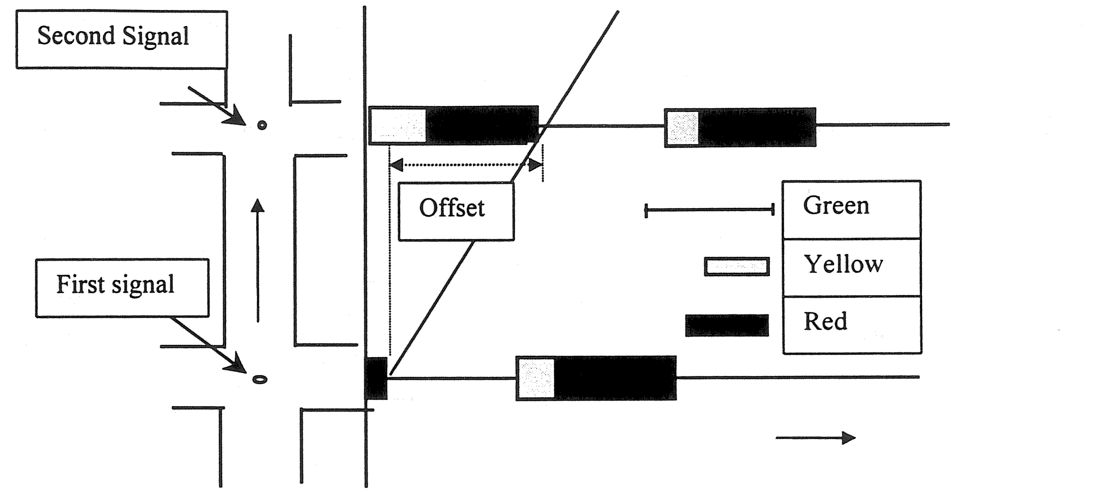


Figure 10: Offset Diagram

4.4 ACCIDENT ANALYSIS

The accident data was collected by ITD during 1990 to 1996. Three hundred and four accidents were observed in the corridor. Of these 304 accidents, one fatality occurred. The highest number of accidents occurred at the intersections of Appleway and Lincoln. Table 6 shows the number and location of accidents. Forty-four accidents per year occur for the entire arterial from Ironwood to Hayden shown in Figure 11.

Table 6. Accident Rates

Location	No. of accidents
Ironwood Dr. & Lincoln way	37
Appleway Ave. & EB Off ramp	5
EB Off ramp & Lincoln way	8
Appleway Ave. & Lincoln way	86
Lincoln way & Neider Ave.	31
Appleway Ave. & Kathleen Ave.	37
Dalton Ave. & Lincoln way	21
Hanley Ave. & Lincoln way	26
Canfield Ave. & Lincoln way	18
Prairie Ave. & US-95	3
Hayden Ave. & Non Applicable	32

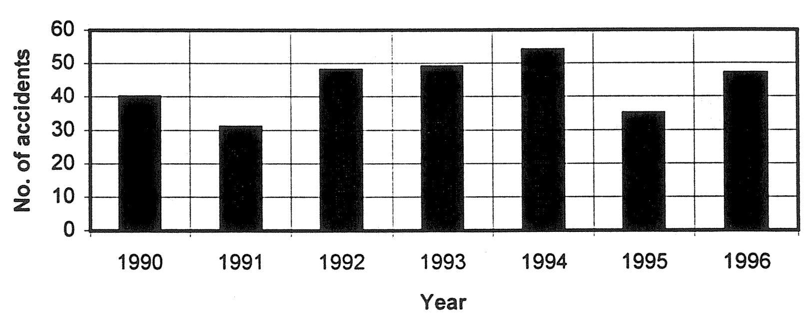


Figure 11. Number of Accidents Recorded Each Year

4.5 ORIGIN-DESTINATION STUDY

An Origin-Destination (O-D) study was conducted to estimate the proportion of the through vehicles exiting at each intersection of the arterial. The O-D data was collected during two peak hours, 12:00 - 1:00 p.m. and 4:15 - 5:15 p.m. on 15th August 1997. The main objective of the study was to determine if sub-systems were needed. For example, if the O-D study showed that a majority of the motorists travel between Ironwood and Appleway, then it may be beneficial to divide the system into sub-systems so a wider bandwidth can easily be obtained. On the other hand, if the majority of vehicles travel between Ironwood and Hayden, then it is better to provide one continuous progression along the arterial.

White vehicles were chosen for the O-D study because there is a high proportion of them compared to other vehicles. All the northbound through white vehicles for two peak periods were recorded. For the mid-day period, a total of 107 vehicles entered the corridor at Ironwood and exited at various intersections throughout the corridor. For the afternoon period, 93 vehicles entered the corridor at Ironwood intersection and exited at various intersections along the arterial. The percentage of vehicles exiting at each intersection was calculated. The percentage of vehicles exiting at each intersection for both mid-day and PM periods are shown in Figure 12. The percentage of vehicles traveled in a particular segment originating at Ironwood for both mid-day and PM periods are shown in Figure 13. It is observed that most of the vehicles originating at Ironwood are exiting at westbound ramp.

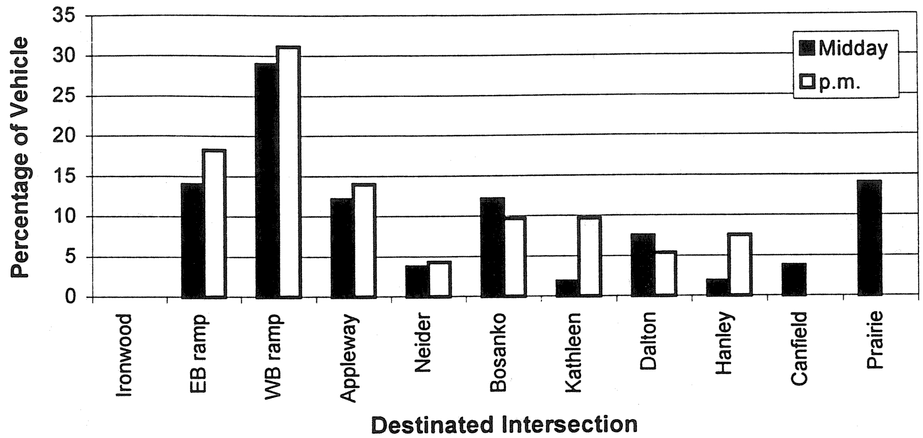


Figure 12. Percentage of Vehicles Destined at Particular Intersection

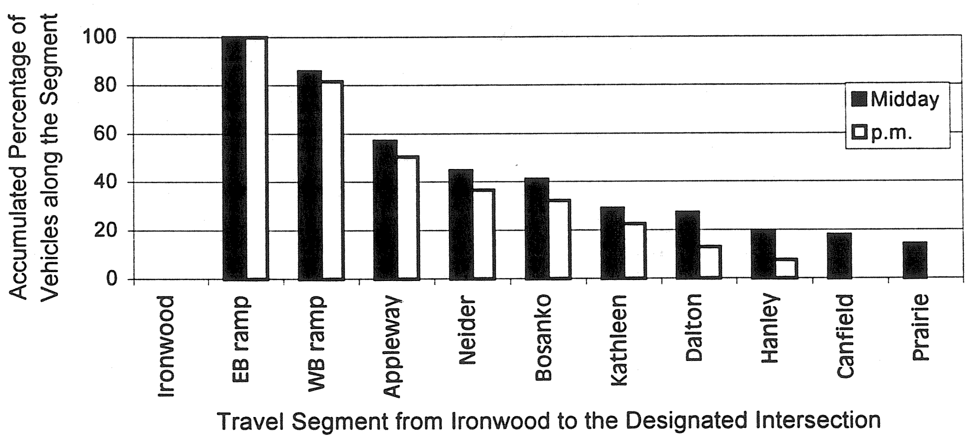


Figure 13. Traffic Flow Between Segments

4.6 SUMMER TRAFFIC VOLUME

Summer data was collected by ITD from July 17, 1997 to July 31, 1997 at Appleway, Bosanko, Dalton, Hanley, Ironwood, Neider, and Prairie intersections. Table 7 shows the summary of the October and July vehicle volume data. Then, the percentage change of volume is calculated and presented in Table 8. Positive percentages indicate the increase in the vehicle volume in summer and negative percentages indicate the decrease in the vehicle volume.

Table 7. Arterial Traffic Volumes (vph)

Cross Street	FALL						SUMMER					
Name	NB	SB	NB + SB	EB	WB	EB + WB	NB	SB	NB + SB	EB	WB	EB + WB
Appleway	1492	1147	2639	572	835	1407	1659	1286	2945	611	860	1471
Dalton	1468	1230	2698	204	198	402	1564	1328	2892	259	253	512
Hanley	1405	1019	2424	197	609	806	1366	1195	2561	167	531	698
Ironwood	677	947	1624	845	654	1499	807	926	1733	983	605	1588
Neider	1221	1303	2524	228	392	620	1408	1444	2852	204	457	661
Prairie	1317	935	2252	295	186	481	1290	962	2252	414	618	1032

Table 8. Change in the Traffic Volumes over the Summer

Cross Street Name	Cross Street Traffic	Main Street Traffic	Overall Intersection Traffic
Appleway	5%	12%	9%
Dalton	27%	7%	10%
Hanley	-13%	6%	1%
Ironwood	6%	7%	6%
Neider	7%	13%	12%
Prairie	115%	0%	20%

5.0 TRAFFIC CONTROL STRATEGIES

There are many methodologies which can be implemented to improve traffic operations and reduce traffic delay at intersections, for example, changing existing road geometry by adding additional lanes to increase capacity, or dividing lanes to separate passenger cars and heavy vehicles to avoid mixed vehicle conflicts. However, one of the least expensive methods is coordinating the timing of traffic signals to keep delays to a minimum. To do this, engineers must adjust the duration of each complete traffic timing cycle and the synchronization of a series of traffic lights to maximize the number of drivers who can pass a succession of greens without stopping on red signals. Studies have found that travel time, delays, vehicle stops and fuel consumption can be reduced substantially if signalized intersections are timed properly (i.e., the signal time settings are optimized to minimize delays and stops at all approaches of intersections). Many of these studies reported a range of benefit-to-cost ratios as high as 100-to-1. In addition, reductions in travel time, stops, delays, and fuel consumption also have been reported at a range of 10 to 40 percent (1). Various computer models are used in the analysis and are discussed below.

5.1 COMPUTER MODELS

5.1.1 Highway Capacity Software (HCS)

HCS was applied to evaluate the level of services for each isolated intersection for EXISTING and PROPOSED signal timing settings. Since HCS does not have optimization function, signal timings must be adjusted manually to reach a desire level of service. For evaluation of EXISTING signal timing, the cycle length was set to 120 seconds and a low pedestrian volume was set at 50 pedestrian per hour. The LOS for existing and modified conditions is summarized and is shown in Appendix G.

5.1.2 SIDRA

SIDRA is an Australian model that is generally used as an aid for design and evaluation of different types of intersections, including roundabouts. The capability of SIDRA model includes evaluation and optimization of capacity and performance in terms of delay and queue length in each isolated intersection. Both LOS and delay were observed for each approach at every intersection and is shown in Appendix H.

5.1.3 TEAPAC

TEAPAC, the Traffic Engineering Applications Package, is a unique interface. Each program within TEAPAC shares a common user commands, and uses the same menu and full-screen input/editing methods. Thus, one set of input data can be shared within TEAPAC without re-entry data (2).

TEAPAC consists of SIGNAL94, NOSTOP, PREPASSR, PRETRANSYT, PRENETSIM, SITE, TURNS, TED, WARRANTS, and TUTOR. In this study, the primary focuses are on SIGNAL94, PREPASSR, PRETRANSYT, and PRENETSIM. These four programs are described as follows:

5.1.4 SIGNAL94 / TEAPAC

SIGNAL94 is designed to aid in the analysis and optimized design of isolated intersection control based on factors such as approach capacity, lane usage, phasing and pedestrian constraints. The capacity analysis used in SIGNAL94 is based on 1994 Highway Capacity Manual. SIGNAL94 has the capability to optimize cycle length, phase sequences, splits, and level of service (LOS). The model also calculates maximum queues on all intersection approaches, as well as other MOEs such as stops and fuel consumption. The output of SIGNAL94 provides a series of best phasing plans, which allows users to choose the appropriate signal-timing plan. One of the advantages of SIGNAL94 is that it can optimize signal phasing and timing without initial signal phasing and timing data. In addition, the input and output data of SIGNAL94 can be used directly by PREPASSR, PRETRANSYT, and PRENETSIM.

5.1.5 PREPASSR / TEAPAC and PASSER II-90

PREPASSR is designed to aid the use of the PASSER II-90 arterial signal optimization. PREPASSR is an interactive preprocessor program that is used to prepare input data in a fixed format for the PASSER II-90 program. PREPASSR has the ability to read SIGNAL94 data files directly, eliminating the need to re-enter the existing data of SIGNAL94.

PASSER II-90 is an arterial-based progression bandwidth optimizer, which optimizes the offsets of the coordinated phases. In addition, PASSER II-90 calculates the optimum splits by giving equal consideration to traffic flows on the arterial and cross streets.

5.1.6 PRETRANSYT / TEAPAC and TRANSYT-7F

PRETRANSYT is a pre- and post-processor used with the TRANSYT-7F program. PRETRANSYT reads SIGNAL94 data files directly without data re-entry. TRANSYT-7F requires a rigid input stream of specially numbered card types and coded input. TRANSYT-7F simulates the existing signal system, as well as optimizes operations. The main objective of TRANSYT-7F is to minimize both delay and stops of the network, and maintain a good progression on the arterial with special attention to heavy turning movement within the system. Overall, PRETRANSYT allows quick and effective use of TRANSYT-7F.

5.1.7 PRENETSIM / TEAPAC and TRAF-NETSIM

PRENETSIM is a very cost-effective tool. It imports data files directly from SIGNAL94, PREPASSR, PRETRANSYT, or any other TEAPAC programs. However, one disadvantage of PRENETSIM is that it does not include an editor for actuated control cards, Thus, manual input of actuated control cards is required.

TRAF-NETSIM is a microscopic simulation that can be used to simulate traffic operations for arteries, isolated intersections and/or networks. The program supports fixed-time and actuated-controlled intersections. TRAF-NETSIM has a numerous of MOEs that are calculated by movements and on a lane-by-lane basis for all intersection approaches. These MOEs include delays, queue length, queue time, percent-stops, stop-time, travel time, speeds and many other congestion-based measures.

5.1.8 CORSIM

CORSIM is an updated version of TRAF-NETSIM and FRESIM. It is a very sophisticated and powerful microscopic traffic simulation model designed for simulating corridor traffic flow (freeways and surface streets). CORSIM simulates traffic behavior at the microscopic level of the individual vehicle and its interaction of with surrounding vehicles. However, reasonable understanding of the assumptions, theories, and logic of the model are required in order to fully benefit from the software. Further understanding of the model can be obtained from a recent article, "CORSIM-Corridor Traffic Simulation Model" written by Abolhassan Halati, Henry Lieu, and Susan Walker (3).

5.2 INPUT DATA

Input data for this study is summarized in Appendix I-I, including traffic volume, number and width of lane, saturation flow rate, and existing signal timing. The Friday PM peak 15-minute volume, peak hour factor of 1.0, and ideal saturation flow of 1900 vehicle per hour per lane (vphpl) are used for the analysis.

5.2.1 Procedure

The procedures used to obtain the MOEs for EXISTING, OPTIMIZED, and PROPOSED signal timing are described below:

5.2.2 EXISTING signal timing

EXISTING signal timing for each isolated intersection is obtained based on the field data observed in October 1996. The signal timings are further evaluated on SIGNAL94 and HCS for the level of service. Then TEAPAC is used to code PRENETSIM model and updated to CORSIM model. Further calibrations of CORSIM input file is required to reflect the field conditions and to ensure its accuracy. The calibrations includes pocket length, merging lane, extend link length, pedestrians volume, headway, and aggressiveness of motorists. Finally, the calibrated model is simulated for its measure of effectiveness.

5.2.3 OPTIMIZED signal timing

OPTIMIZED signal timing is obtained from the optimization of EXISTING signal timing with SIGNAL 94. The resulting of OPTIMIZED signal timing is then evaluated and simulated in CORSIM for its measure of effectiveness.

5.2.4 PROPOSED signal timing

PROPOSED signal timing used Friday peak hourly volume and peak hour factor of 0.95 for all intersections. The development of PROPOSED signal-timing plan included the following steps:

- Code each intersection in SIGNAL94 to yield saturation flow rates.
- Create PASSER II-90 and TRANSYT-7F input files for existing signal timing and volumes.
- Calibrate to existing condition using TRANSYT-7F and existing signal timings.
- Determine phasing sequence using PASSER II-90.
- Optimize splits and offsets using TRANSYT-7F.
- Analyze sub-system and define progression boundaries in TRANSYT-7F.
- Modify phasing splits considering pedestrian crossing requirements.
- Fine tuning offsets and phasing to consider partial green bands.
- Identify phasing changes from existing phasing to meet controller capabilities.
- Simulate signal timing using CORSIM.

5.3 ANALYSIS and EVALUATION

Analysis of individual intersections was conducted using the HCS and SIGNAL94. The results of HCS analysis are summarized in Table 9.

Table 9. HCS Result of EXISTING and PROPOSED for Each Intersection

Intersection	EXISTING			PROPOSED		
	Critical v/c	Delay (sec/veh)	LOS	Critical v/c	Delay (sec/veh)	LOS
Hayden	>1.0	>60	F	0.79	27.5	D
Prairie	>1.0	>60	F	0.66	14.8	B
Canfield	>1.0	>60	F	0.62	15.8	C
Hanley	>1.0	>60	F	0.79	24.5	C
Dalton	0.72	25.8	D	0.62	14.5	B
Kathleen	0.68	24.0	C	0.72	21.0	C
Bosanko	~	~	~	0.61	14.0	B
Neider	0.72	27.0	D	0.64	17.1	C
Appleway	>1.0	>60	F	0.80	27.2	D
WB Ramp	>1.0	>60	F	0.58	11.1	B
EB Ramp	0.90	34.4	D	0.73	17.8	C
Ironwood	>1.0	>60	F	0.77	26.4	D

The EXISTING, OPTIMIZED, and PROPOSED level of services and signal phasing and timing for each intersection are shown in Appendix I-I based on SIGNAL94 analysis.

The analysis is based on the measure of effectiveness from CORSIM output for EXISTING, OPTIMIZED, and PROPOSED signal timing settings including: 1) comparison of through speed, 2) comparison of delay time, and 3) comparison of queue length for each movement. The analyses of the MOEs are based on individual links along the arterial. The depiction shown in Appendix I-II illustrates the link diagram that is used throughout the analysis. There are a total of 16 links and 17 nodes: nodes 750 and 720 are entry nodes, node 45 is dummy node, and node 2 and 4 are stop control intersections. A link is defined as the segment of roadway connecting two nodes.

The CORSIM measure of effectiveness is shown in Appendix I-II (Table II-1 and II-2), including EXISTING, OPTIMIZED, and PROPOSED through speed and delay time of each link for both northbound and southbound traffic directions. Queue lengths for each movement of intersection are also shown in Appendix I-II (Table II-3).

5.3.1 Comparison of Through Speed

According to Appendix I-II (Table II-1) and Appendix I-III (Figure III-1). The PROPOSED signal timings show significant improvement after TRANSYT-7F optimization over EXISTING and OPTIMIZED, except for a slight decline in links 1, 4, 5, 6, and 13 in the northbound direction, and decline on links 1, 9, and 13 in the southbound direction. The results show that minimizing delays can greatly improve corridor speed. Minor through-speed declines are due to the adjustments of the signal timing as a result of optimization.

5.3.2 Comparison of Delay Time

According to Appendix I-II (Table II-2) and Appendix I-III (Figure III-2). The PROPOSED have significant reduction in delay for the entire arterial over EXISTING and OPTIMIZED except a slight delay increased on links 6 and 13 in the northbound direction, and on links 1, 6, 9, and 13 in the southbound direction. As expected, the results show that reduction in delay time has agreed with the objective function of TRANSYT-7F of minimizing delays.

5.3.3 Comparison of Queue Length

According to Appendix I-II (Table II-3) and Appendix I-III (Figure III-3), the PROPOSED signal setting has significantly decreased queue length for through movement, except a slight queue in link 7 and 10. Right-turn queue appeared to be fine for all cases, EXISTING, OPTIMIZED, and PROPOSED. One vehicle queue at link 6 of PROPOSED appeared to be a minor problem. Such phenomenon is primary due to random seed values during simulation. PROPOSED left-turn queue length appeared to be fairly low throughout the arterial, except a significant queue at link 7. This can be explained by platoon dispersion, where vehicles begin to disperse along the long distance between intersections and some vehicles were left behind and are stopped by the red light. For southbound THRU queue length in Appendix I-III (Figure III-4), PROPOSED showed fewer queues built up than EXISTING and OPTIMIZED except link 8 and 13. No PROPOSED right-turn queue appeared in the simulation throughout the arterial, which indicated that the demand is well under the capacity. Southbound left-turn queue seems to be a problem for OPTIMIZED and PROPOSED even after optimization. Keep in mind that left-turn and THRU are conflicting movements. In order to provide long progression bandwidth for the arterial that most green timing has to be allocated to the priority THRU movement instead of left-turns.

5.3.4 Improvement of THRU Speed, Delay Time, and Queue Length

Figures in Appendix I-IV consist of percent improvement of northbound and southbound traffic flow through speed, delay time, and queue length of EXISTING and PROPOSED signal timings. The calculation of through speed, delay time, and queue length improvements based on each link are shown in Appendix I-IV (Figures IV-1, IV-2, and IV-3) respectively. The percentage of speed improvement is calculated based on the following equation:

$$\% \text{ Improvement} = \frac{(\text{EXISTING} - \text{PROPOSED}) \times 100}{\text{EXISTING}}$$

The percent improvement of delay time and queue length is calculated based on the following equation:

$$\% \text{ Improvement} = \frac{-\text{(EXISTING} - \text{PROPOSED)} \times 100}{\text{EXISTING}}$$

Percent improvement of THRU speed in Appendix I-IV (Figure IV-1), percent improvement of THRU delay time in Appendix I-IV (Figure IV-2), and percent improvement of THRU queue length in Appendix I-IV (Figure IV-3) all have shown dramatic improvement in PROPOSED over EXISTING signal settings. However, some links have shown the reverse trend where the speed, delay time, and queue length is worse than the EXISTING signal setting. This reverse trend can be addressed by signal timing adjustment to provide better overall intersection performance and arterial progression.

5.4 RESULTS

The overall result including progression bandwidth and average arterial speed is shown in Table 10.

Table 10. TRANSYT-7F Progression Bandwidth and Average Speed of PORPOSED Signal Timing

	AM		MIDDAY		PM	
	NB	SB	NB	SB	NB	SB
Average Speed (mph)	31.3	34	29.7	30.4	28.8	29.6
Bandwidth (sec)	14	43	26	21	29	21

The signal timing maps are presented in Appendix I-V. There are four maps, one for the existing PM peak period and three for the final proposed signal timing for the PM, AM and Midday peak periods. Each map contains the lane configuration, peak hour volumes, and the signal-timing plan including the cycle length and offsets.

6.0 ACKNOWLEDGEMENT

This research project was founded through the Idaho Transportation Department. The assistance of Sanjeev Tandle, who summarized field data and performed computer model analyses, and Raymond Wallace, who helped in editing are appreciated. The authors acknowledge the valuable advice of Dr. Michael Kyte in completing this research project.

7.0 CONCLUSION AND RECOMMENDATION

The research project reached the following conclusions:

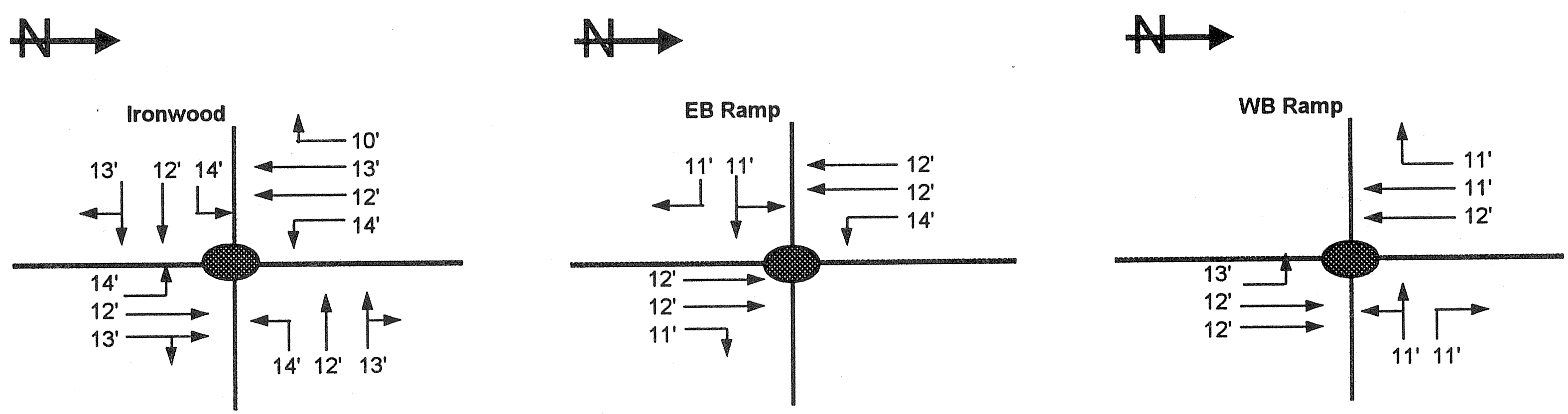
- Based on the collected field traffic volumes, no major capacity deficiencies were found for any the intersection within the arterial. Queuing problems may be due to poor progression.
- The analysis of sub-systems showed that no major benefits were obtained by dividing the system into sub-systems. Although it may be difficult to maintain progression for the intersections at Hayden and Prairie due to the long distance between them, they are included in the system to at least progress those possibly well maintained platoons.
- A cycle length of 115 seconds was found to be the best for all the time periods. Some intersections require phasing modification from the existing phasing. These intersections include Dalton, Kathleen, Appleway, WB ramp, and Ironwood.
- The use of CORSIM was valuable in evaluating timing plans before implementation.

The new signal timings have shown a substantial improvement over the EXISTING signal timing. Based on the comparison of measure of effectiveness from CORSIM, the PROPOSED signal timing is found to be the most effective in solving delay problems in US-95 from Coeur d'Alene to Hayden. The PROPOSED signal timing for AM, Mid-day, and PM are shown in Appendix I-V.

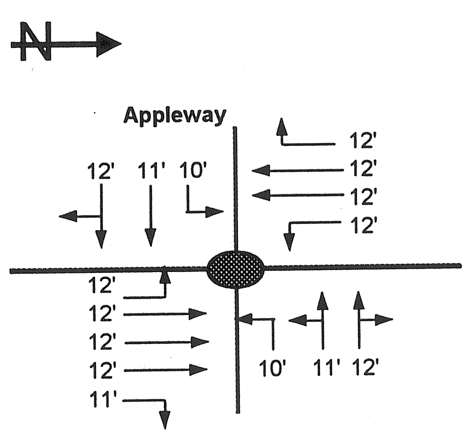
8.0 REFERENCES

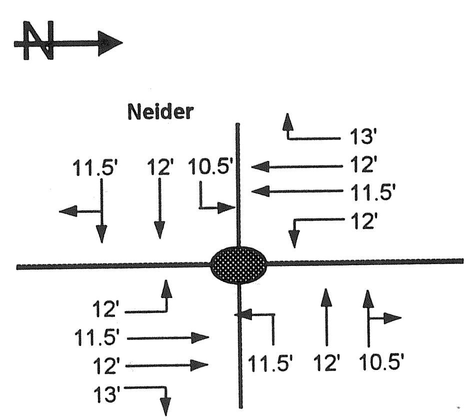
1. Ziad A. Sabra, and Charles R. Stockfisch. Advanced Traffic Models: State-of-the-Art. ITE Journal, September 1995, pp31-42.
2. Strong Dennis W. TEAPAC Manual. Strong Concept. June 1995.
3. Halati Abolhassan, Lieu Henry, and Walker Susan. CORSIM- Corridor Traffic Simulation Model. ASCE. New York, New York.
4. Hesham A. Rakha and Michael W. Van Aerde. Comparison of Simulation Modules of TRANSYT and INTEGRATION Models. Transportation Research Record, No. 1566, pp. 1-7.
5. McShane R. William and Roess P. Roger. Traffic Engineering. Prentice Hall, Englewood Cliffs, New Jersey, 1990,

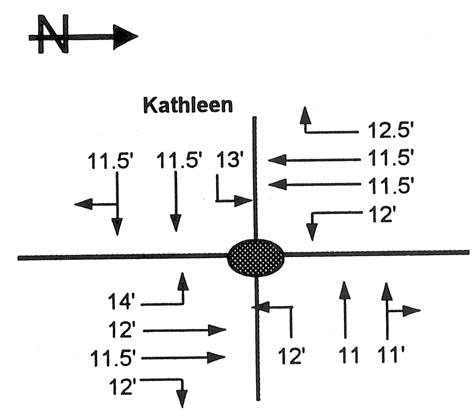
A. Geometric Layout

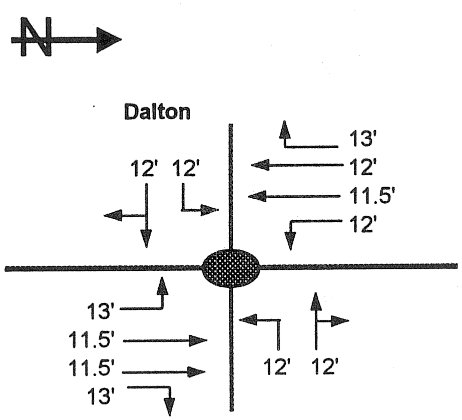


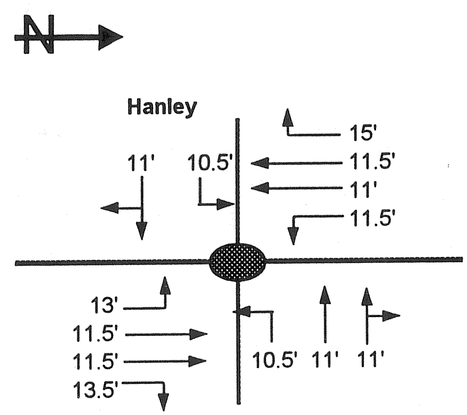
*All lane widths are expressed in feet

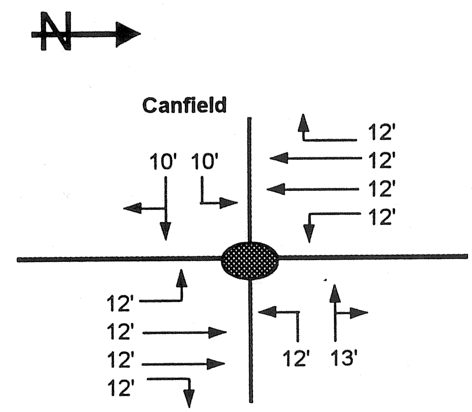

N
Appleway
12'
12'
12'
12'
12'
11'
10'
12'
12'
12'
12'
11'
10'
11'
12'

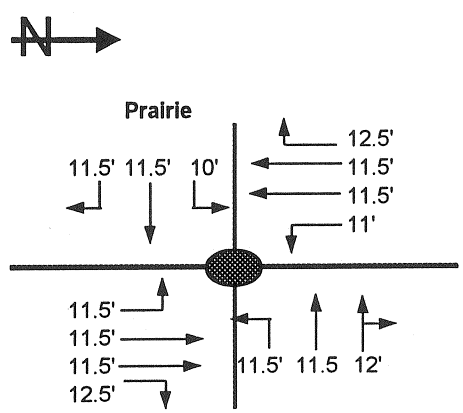

N
Neider
13'
12'
11.5'
12'
11.5'
12'
10.5'
12'
11.5'
12'
13'
11.5'
12'
10.5'

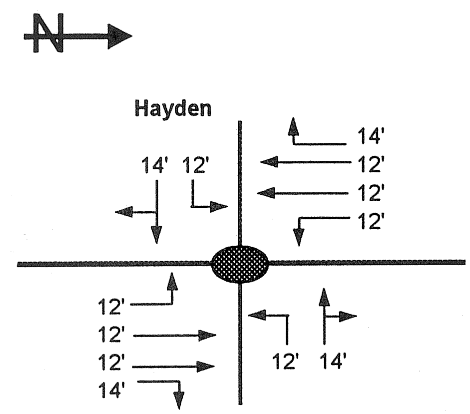

N
Kathleen
12.5'
11.5'
11.5'
12'
11.5'
11.5'
13'
14'
12'
11.5'
12'
12'
11
11'


N
Dalton
13'
12' 12'
12'
11.5'
12'
13'
11.5'
11.5'
13'
12' 12'


N
Hanley
15'
11' 10.5'
11.5'
11'
11.5'
13'
11.5'
11.5'
13.5'
10.5' 11' 11'


N
Canfield
12'
10' 10'
12'
12'
12'
12'
12'
12'
12'
12' 13'


N
Prairie
12.5'
11.5' 11.5' 10'
11.5'
11.5'
11'
11.5'
11.5'
11.5'
12.5'
11.5' 11.5 12'


N
Hayden
14'
14' 12'
12'
12'
12'
12'
12'
12'
14'
12' 14'

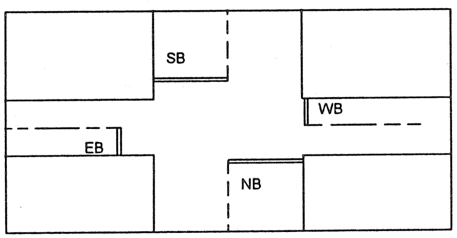
B. Traffic Data Summary

SUMMARY FOR PM PERIOD

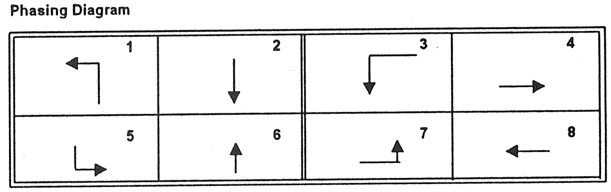
Turning Movement Volumes (vph)

DATE: 10/25/96 **DAY:** Friday
WEATHER Rainy **ANALYST/ RECORDER:** Lin

U.S. 95 & Cross Street	NORTH BOUND				SOUTH BOUND				EAST BOUND				WEST BOUND				TOTAL INTERSECTION VOLUME	Peak Hour	Peak 15-minute Interval	Peak Hour Factor
	LT	TH	RT	Total	LT	TH	RT	Total	LT	TH	RT	Total	LT	TH	RT	Total				
Hayden	77	1022	337	1436	113	531	84	728	91	180	70	341	144	191	242	577	3,082	4:45 - 5:45	5:30 - 5:45	0.94
Praire	86	1209	22	1317	20	850	65	935	99	52	144	295	44	76	66	186	2,733	4:15 - 5:15	4:45 - 5:00	0.98
Canfield	5	1145	170	1320	123	920	3	1046	8	11	4	23	149	4	172	325	2,714	4:15 - 5:15	4:45 - 5:00	0.94
Hanley	299	1078	28	1405	114	868	37	1019	50	130	17	197	235	117	257	609	3,230	4:30 - 5:30	4:45 - 5:00	0.92
Dalton	106	1259	103	1468	56	1106	68	1230	57	60	87	204	63	75	60	198	3,100	4:00 - 5:00	4:45 - 5:00	0.97
Kathleen	141	1159	135	1435	134	1066	108	1308	113	195	177	485	97	184	160	441	3,669	3:30 - 4:30	3:45 - 4:00	0.95
Neider	57	1032	132	1221	119	1056	128	1303	114	48	66	228	98	101	193	392	3,144	3:45 - 4:45	4:00 - 4:15	0.86
Appleway	207	1032	253	1492	212	812	123	1147	110	311	151	572	245	363	227	835	4,046	4:30 - 5:30	5:00 - 5:15	0.91
WB Ramp	235	1207		1442		696	429	1125				0	65		213	278	2,845	4:45 - 5:45	5:00 - 5:15	0.94
EB Ramp		954	123	1077	188	822		1010	371		183	554				0	2,641	3:30 - 4:30	3:45 - 4:00	0.96
Ironwood	106	491	80	677	272	462	213	947	255	478	112	845	83	269	302	654	3,123	3:30 - 4:30	3:30 - 3:45	0.96



Phasing Diagram

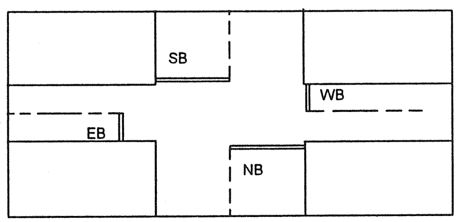


SUMMARY FOR AM PERIOD

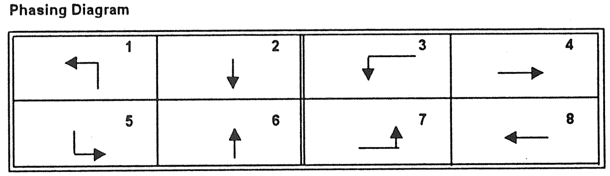
Turning Movement Volumes (vph)

DATE: 10/25/96 **DAY:** Friday
WEATHER Rainy **ANALYST/ RECORDER:** Lin

U.S. 95 & Cross Street	NORTH BOUND				SOUTH BOUND				EAST BOUND				WEST BOUND				TOTAL INTERSECTION VOLUME	Peak Hour	Peak 15-minute Interval	Peak Hour Factor
	LT	TH	RT	Total	LT	TH	RT	Total	LT	TH	RT	Total	LT	TH	RT	Total				
Hayden	42	334	92	468	76	823	86	985	58	84	78	220	119	90	53	262	1,935	7:15-8:15	7:45-8:00	0.87
Praire	39	440	17	496	22	956	75	1053	63	30	115	208	10	38	14	62	1,819	7:15-8:15	7:45-8:00	0.83
Canfield	3	515	35	553	47	990	9	1046	2	0	0	2	24	3	15	42	1,643	7:15-8:15	7:45-8:00	0.85
Hanley	85	479	25	589	94	877	11	982	36	67	29	132	75	58	80	213	1,916	7:30-8:30	7:45-8:00	0.81
Dalton	57	542	72	671	71	876	49	996	30	61	70	161	44	45	32	121	1,949	7:45-8:45	7:45-8:00	0.8
Kathleen	74	540	74	688	93	761	81	935	54	137	114	305	69	140	70	279	2,207	8:30-9:30	9:00-9:45	0.86
Neider	21	589	36	646	47	885	40	972	45	25	20	90	21	19	29	69	1,777	7:45-8:45	7:45-8:00	0.88
Appleway	84	565	162	811	112	617	64	793	70	195	122	387	187	136	108	431	2,422	8:30-9:30	9:00-9:15	0.9
WB Ramp	69	601		670		624	243	867				0	107		157	264	1,801	8:30-9:30	9:00-9:15	0.93
EB Ramp		378	40	418	136	725		861	267		221	488				0	1,767	7:45-8:45	7:45-8:00	0.78
Ironwood	94	247	20	361	141	376	382	899	83	113	36	232	16	237	59	312	1,804	7:45-8:45	8:00-8:15	0.73



Phasing Diagram



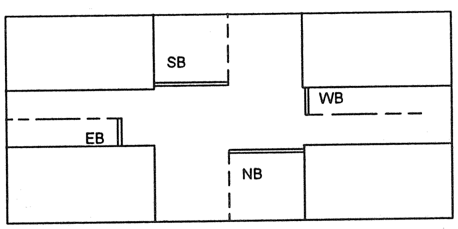
N

SUMMARY FOR MID-DAY PERIOD

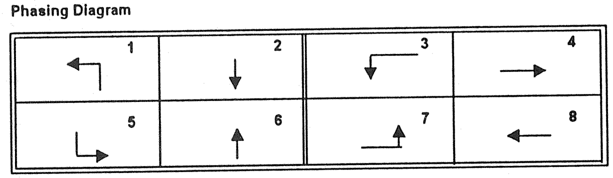
Turning Movement Volumes (vph)

DATE: 10/25/96 **DAY:** Friday
WEATHE Rainy **ANALYST/ RECORDER:** Lin

U.S. 95 & Cross Street	NORTH BOUND				SOUTH BOUND				EAST BOUND				WEST BOUND				TOTAL INTERSECTION VOLUME	Peak Hour	Peak 15-minute Interval	Peak Hour Factor
	LT	TH	RT	Total	LT	TH	RT	Total	LT	TH	RT	Total	LT	TH	RT	Total				
Hayden	81	602	207	890	114	611	70	795	53	121	79	253	156	135	126	417	2,355	2:00-1:00	2:30-12:45	0.94
Praire	66	808	39	913	25	763	60	848	86	32	107	225	32	26	49	107	2,093	2:00-1:00	2:15-12:30	0.93
Canfield	1	845	177	1023	153	799	5	957	3	13	7	23	192	9	145	346	2,349	2:00-1:00	2:15-12:30	0.92
Hanley	290	793	22	1105	108	811	41	960	51	120	21	192	175	122	232	529	2,786	2:00-1:00	12:45-1:00	0.97
Dalton	75	970	98	1143	53	981	54	1088	55	66	84	205	71	82	68	221	2,657	:45-12:45	2:30-12:45	0.94
Kathleen	137	725	127	989	118	939	104	1161	91	151	186	428	115	172	161	448	3,026	2:00-1:00	12:45-1:00	0.85
Neider	51	891	152	1094	110	932	114	1156	93	48	63	204	97	74	115	286	2,740	:30-12:30	2:15-12:30	0.88
Appleway	205	844	321	1370	248	789	100	1137	115	303	154	572	289	281	262	832	3,911	2:00-1:00	2:15-12:30	0.94
WB Ramp	150	1045		1195	333	806		1139				0	97		208	305	2,639	:45-12:45	1:45-12:00	0.85
EB Ramp		908	91	999	175	789		964	360		143	503				0	2,466	2:00-1:00	2:15-12:30	0.94
Ironwood	110	435	67	612	264	417	195	876	235	418	103	756	71	296	279	646	2,890	2:00-1:00	2:15-12:30	0.97



Phasing Diagram



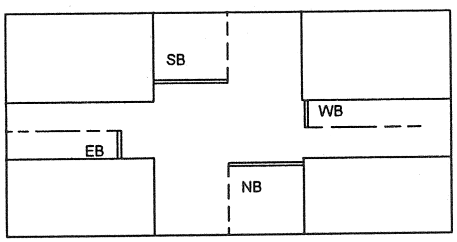
SUMMARY FOR PM PERIOD

Turning Movement Volumes (vph)

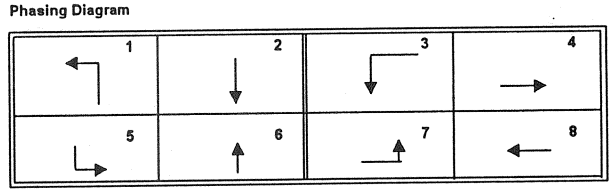
DATE: 10/24/96
WEATHER: Rainy

DAY: Thursday
ANALYST/ RECORDER: Sanjeev Kumar Tandle

U.S. 95 & Cross Street	NORTH BOUND				SOUTH BOUND				EAST BOUND				WEST BOUND				TOTAL INTERSECTION VOLUME	PM Peak Hour	Peak 15-minute Interval	Peak Hour Factor
	LT	TH	RT	Total	LT	TH	RT	Total	LT	TH	RT	Total	LT	TH	RT	Total				
Hayden	95	840	265	1200	107	526	58	691	91	172	72	335	142	154	203	499	2,725	4:30 - 5:30	4:30 - 4:45	0.93
Praire	96	1048	25	1169	35	818	77	930	119	63	165	347	54	64	89	207	2,653	4:00 - 5:00	4:30 - 4:45	0.88
Canfield	3	1173	137	1313	124	867	2	993	12	5	3	20	163	4	119	286	2,612	4:15 - 5:15	4:45 - 5:00	0.94
Hanley	243	1068	44	1355	89	828	53	970	35	101	28	164	190	111	235	536	3,025	4:30 - 5:30	4:30 - 4:45	0.97
Dalton	99	1149	107	1355	50	1041	87	1178	55	82	79	216	75	97	71	243	2,992	4:15 - 5:15	4:15 - 4:30	0.90
Kathleen	105	1129	94	1328	103	987	62	1152	101	158	154	413	83	157	174	414	3,307	4:30 - 5:30	5:00 - 5:15	0.96
Neider	50	1060	108	1218	166	945	96	1207	87	52	57	196	104	76	138	318	2,939	3:45 - 4:45	4:30 - 4:45	0.88
Appleway	158	889	223	1270	187	792	131	1110	94	268	142	504	268	321	179	768	3,652	4:00 - 5:00	4:30 - 4:45	0.92
West Ramp	150	944		1094		867	464	1331					127		225	352	2,777	3:30 - 4:30	3:30 -3:45	0.89
East Ramp		887	128	1015	188	688		876	376		179	555					2,446	3:45 - 4:45	3:45 - 4:00	0.94
Ironwood	111	455	74	640	236	388	175	799	252	417	104	773	62	222	239	523	2,735	3:45 - 4:45	4:00 - 4:15	0.91



Phasing Diagram



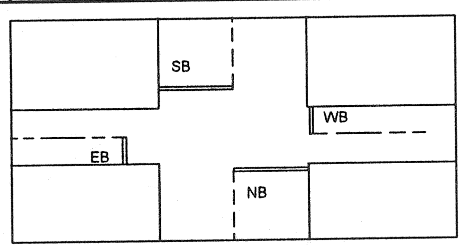
SUMMARY FOR AM PERIOD

Turning Movement Volumes (vph)

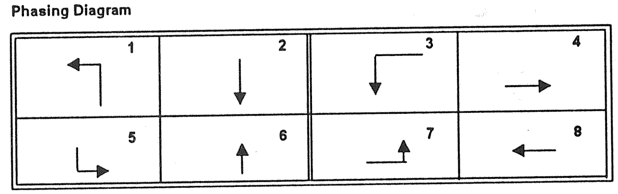
DATE: 10/24/96
WEATHER: Rainy

DAY: Thursday
ANALYST/ RECORDER: Sanjeev Kumar Tandle

U.S. 95 & Cross Street	NORTH BOUND				SOUTH BOUND				EAST BOUND				WEST BOUND				TOTAL INTERSECTION VOLUME	AM Peak Hour	Peak 15-minute Interval	Peak Hour Factor
	LT	TH	RT	Total	LT	TH	RT	Total	LT	TH	RT	Total	LT	TH	RT	Total				
Hayden	40	363	88	491	55	805	104	964	60	107	67	234	124	102	66	292	1,981	7:30 - 8:30	7:45 - 8:00	0.83
Praire	39	410	22	471	50	906	86	1042	86	906	86	1078	7	36	14	57	2,648	7:15 - 8:15	7:45 - 8:00	0.89
Canfield	1	548	51	600	3	894	60	957	2	4	2	8	31	0	18	49	1,614	8:15 - 9:15	8:45 - 9:00	0.89
Hanley	16	477	73	566	74	898	11	983	30	76	22	128	52	63	68	183	1,860	7:30 - 8:30	7:45 - 8:00	0.81
Dalton	44	556	56	656	88	818	36	942	34	46	58	138	33	82	44	159	1,895	7:30 - 8:30	7:30 - 7:45	0.88
Kathleen	71	521	64	656	82	746	94	922	46	121	92	259	58	123	64	245	2,082	8:30 - 9:30	8:45 - 9:00	0.91
Neider	15	593	64	672	42	748	43	833	48	23	30	101	33	24	32	89	1,695	8:30 - 9:30	8:45 - 9:00	0.93
Appleway	71	505	140	716	102	641	62	805	55	153	117	325	154	127	116	397	2,243	8:15 - 9:15	9:00 - 9:15	0.96
West Ramp	68	526		594		647	223	870					142		170	312	1,776	8:15 - 9:15	8:45 - 9:00	0.92
East Ramp		413	42	455	111	711		822	227		132	359					1,636	8:15 - 9:15	8:45 - 9:00	0.83
Ironwood	21	211	63	295	109	333	307	749	98	166	43	307	81	258	20	359	1,710	8:15 - 9:15	9:00 - 9:15	0.82



Phasing Diagram



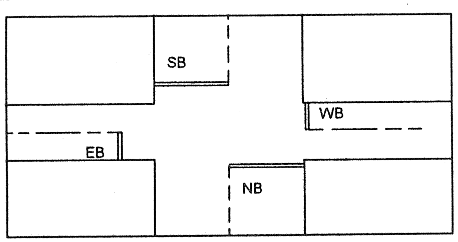
SUMMARY FOR MID-DAY PERIOD

Turning Movement Volumes (vph)

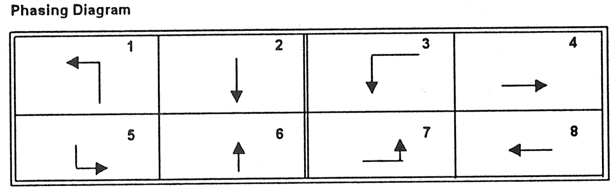
DATE: 10/24/96
WEATHER: Rainy

DAY: Thursday
ANALYST/ RECORDER: Sanjeev Kumar Tandle

U.S. 95 & Cross Street	NORTH BOUND				SOUTH BOUND				EAST BOUND				WEST BOUND				TOTAL INTERSECTION VOLUME	Noon Peak Hour	Peak 15-minute Interval	Peak Hour Factor
	LT	TH	RT	Total	LT	TH	RT	Total	LT	TH	RT	Total	LT	TH	RT	Total				
Hayden	68	490	158	716	79	517	60	656	52	120	75	247	134	122	107	363	1,982	12:00 - 1:00	12:00 - 12:15	0.97
Praire	71	700	30	801	19	726	86	831	81	41	130	252	22	28	28	78	1,962	11:15 - 12:15	12:00 -12:15	0.81
Canfield	10	754	176	940	125	681	8	814	6	23	2	31	171	9	117	297	2,082	11:45 - 12:45	12:15 - 12:30	0.93
Hanley	261	711	15	987	163	762	27	952	52	125	27	204	187	121	225	533	2,676	12:00 - 1:00	12:00 - 12:15	0.89
Dalton	68	881	80	1029	45	867	60	972	46	62	67	175	48	48	54	150	2,326	11:30 - 12:30	12:15 - 12:30	0.92
Kathleen	124	990	125	1239	84	805	92	981	78	147	137	362	80	170	141	391	2,973	11:15 - 12:15	11:15 - 11:30	0.83
Neider	44	811	120	975	94	845	89	1028	72	40	49	161	96	58	126	280	2,444	12:00 - 1:00	12:00 - 12:15	0.94
Appleway	150	691	316	1157	183	631	100	914	74	270	151	495	255	275	234	764	3,330	11:45 - 12:45	12:30 - 12:45	0.95
West Ramp	102	938		1040		666	298	964					98		136	234	2,238	11:45 - 12:45	12:15 - 12:30	0.93
East Ramp		817	86	903	136	693		829	308		100	408					2,140	11:45 - 12:45	11:45 - 12:00	0.93
Ironwood	95	382	69	546	237	329	192	758	204	362	89	655	60	263	227	550	2,509	12:00 - 1:00	12:00 - 12:15	0.88



Phasing Diagram



C. Mid-block Volume

BALANCING DATA TO GET THE MID-BLOCK VOLUMES FOR PEAK 15 MINUTES

DATE:	10/25/96	**DAY:**	Friday
WEATHER:	Rainy	**RECORDER:**	Sanjeev Kumar Tandle
Time - Period analyzed:	PM	**Time:**	5:00 - 5:15

Ironwood

	206	
25	113	68
R	T	L

	30	L
211	165	T
	16	R

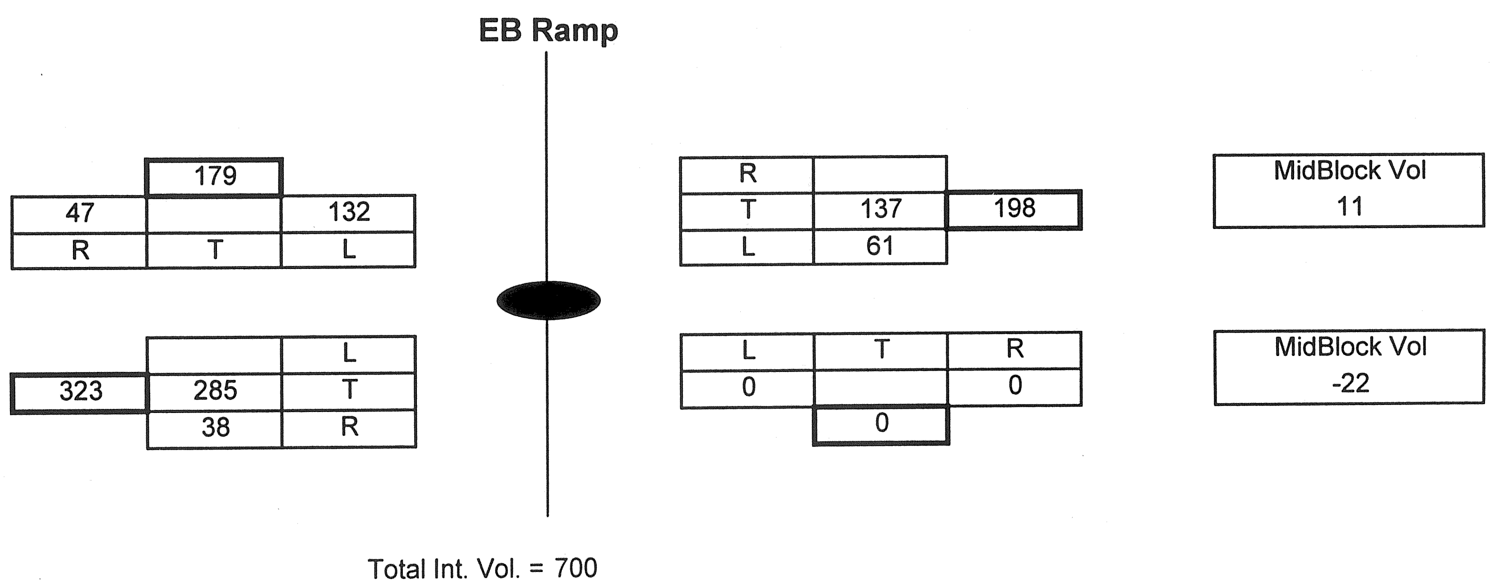
R	22	
T	104	193
L	67	

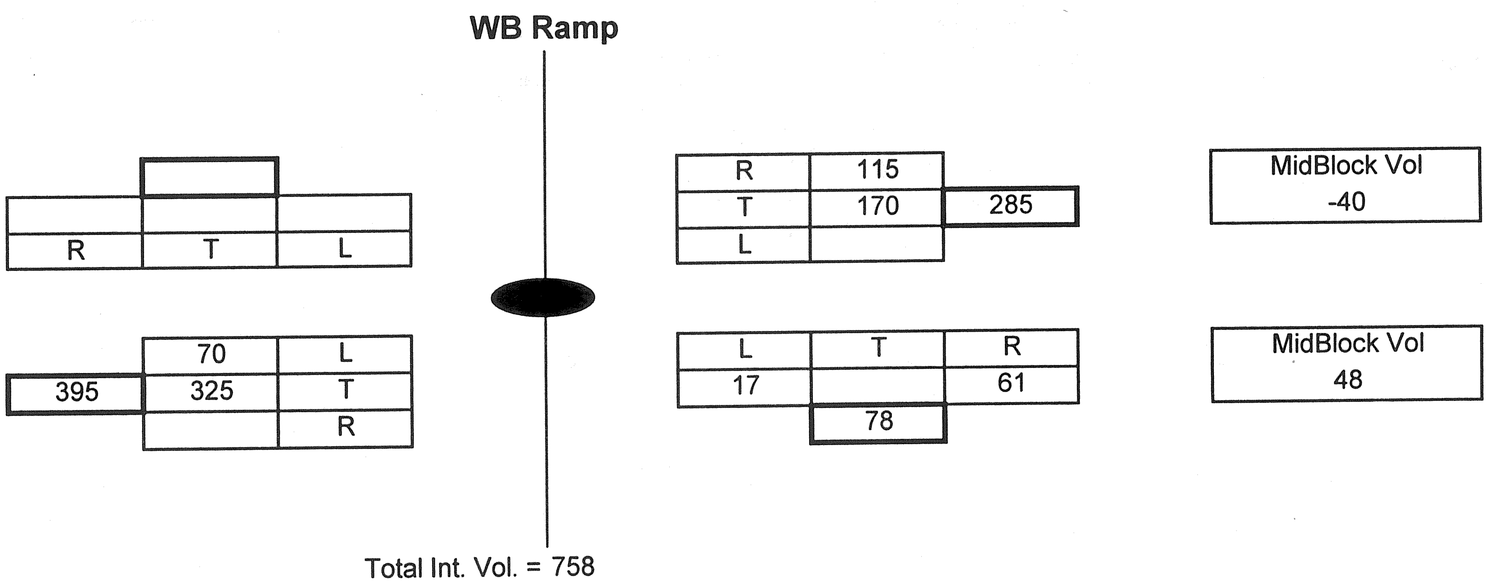
L	T	R
22	45	87
	154	

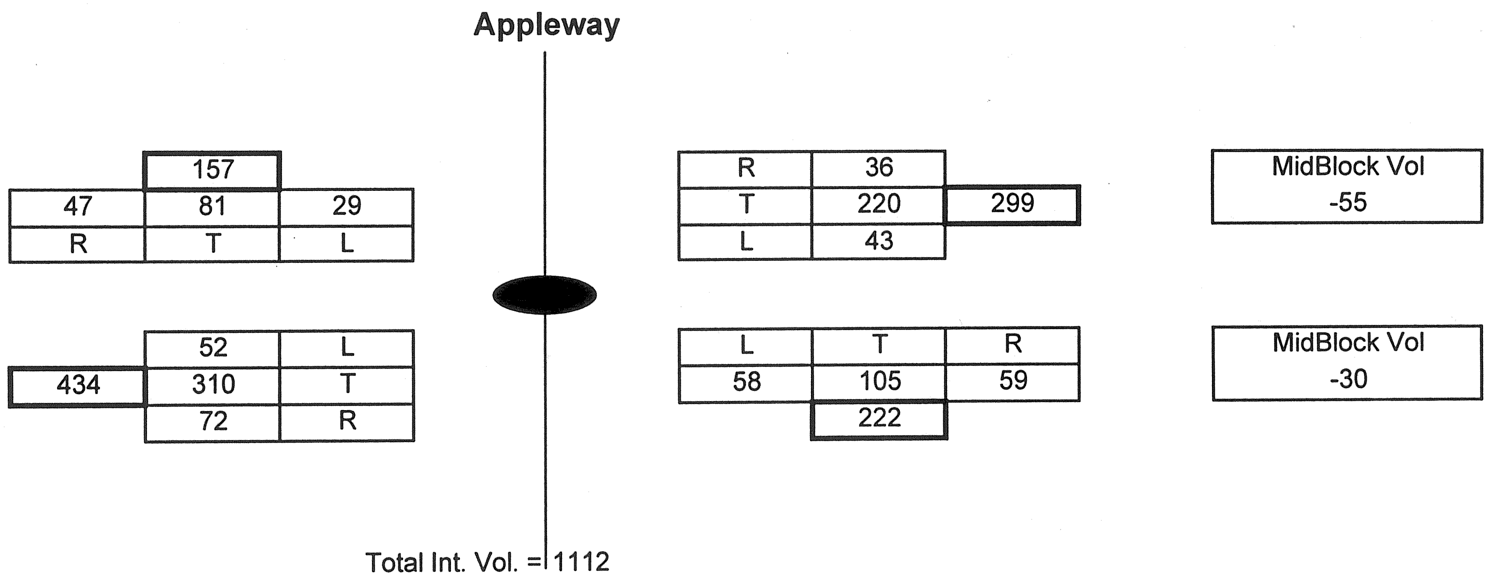
MidBlock Vol
9

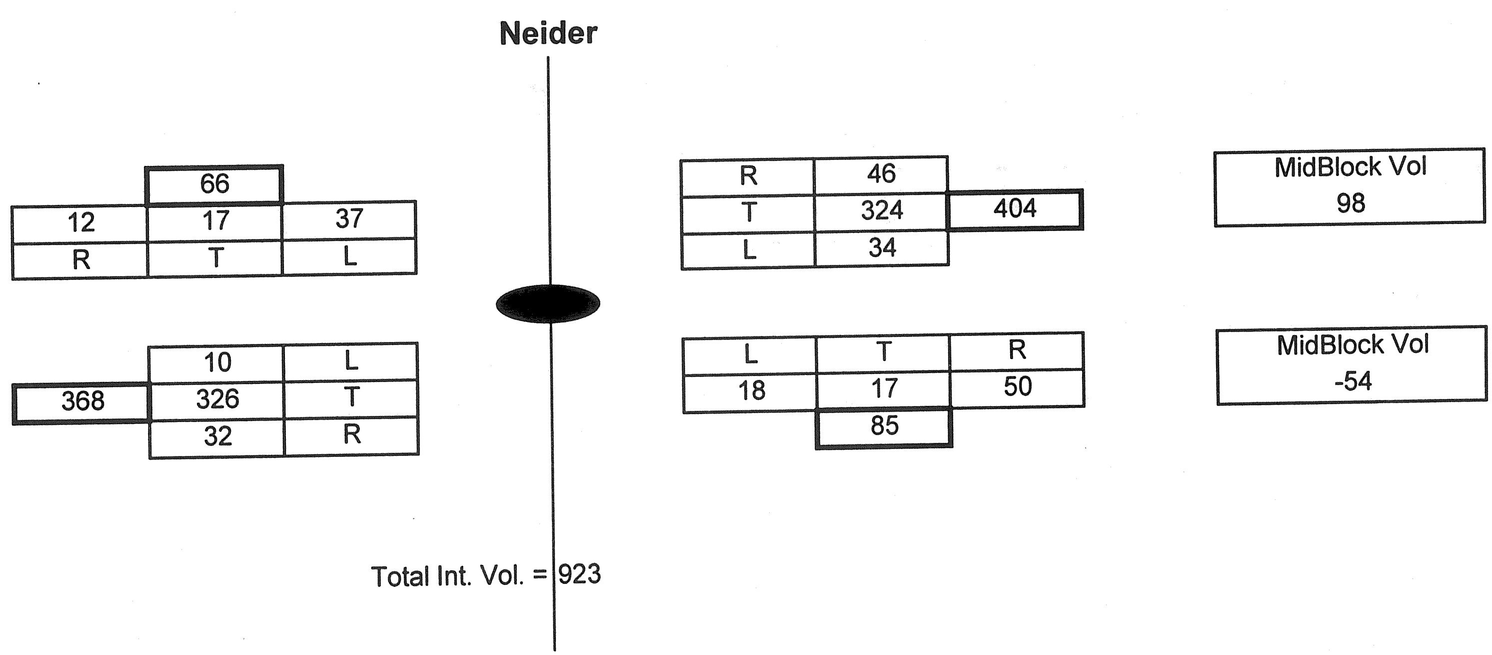
MidBlock Vol
3

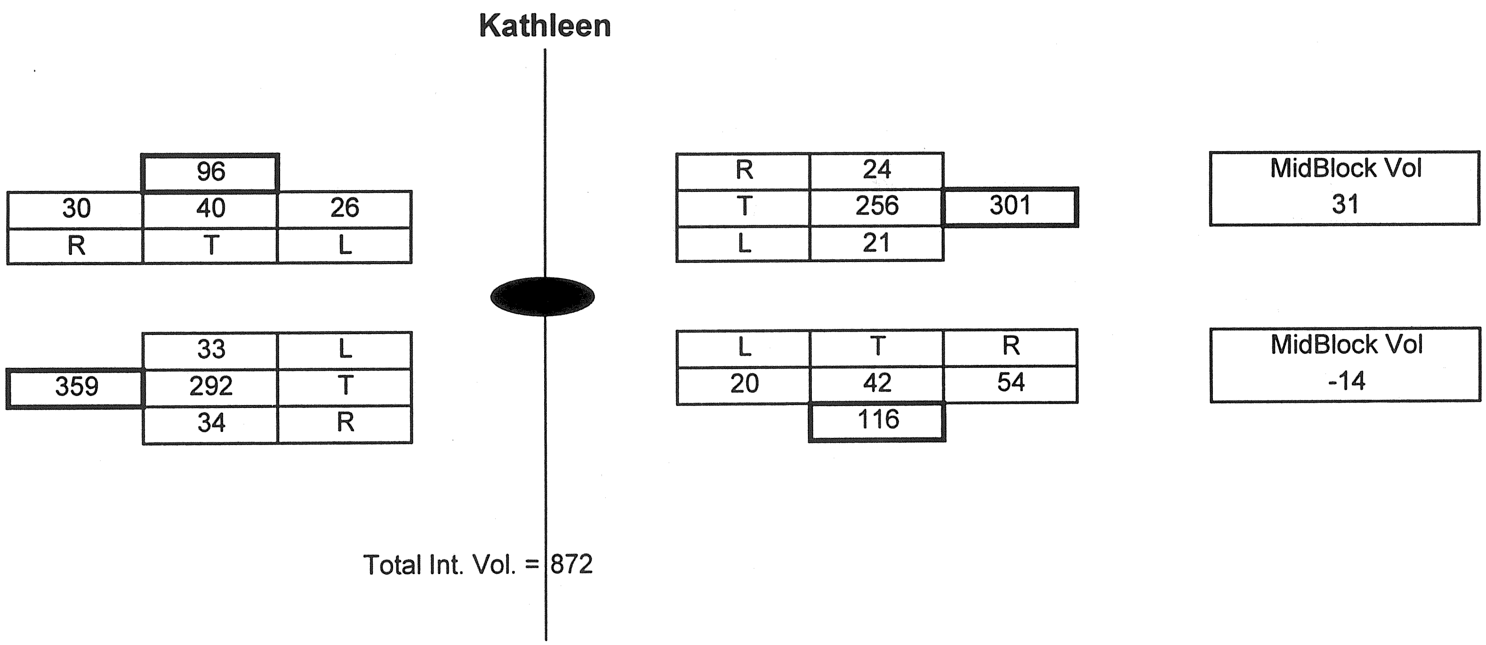
Total Int. Vol. = 764

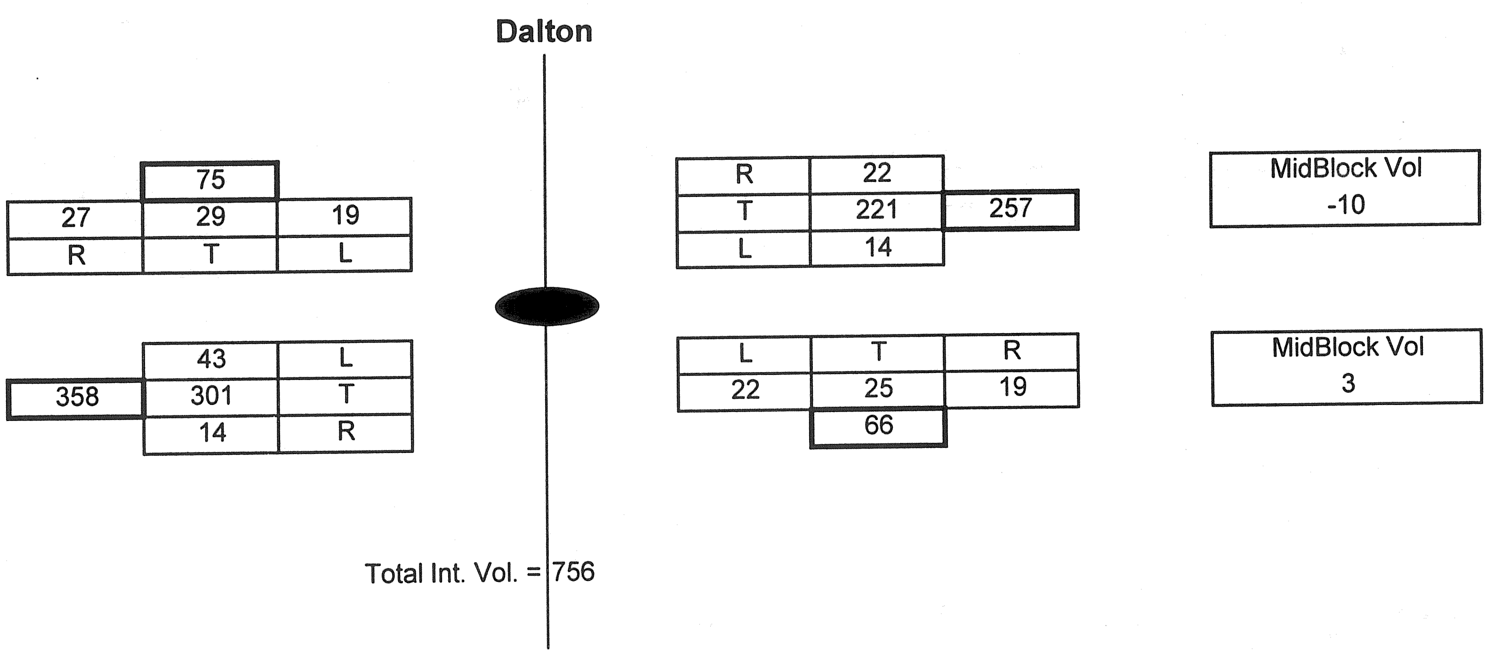

EB Ramp
179
47
132
R
T
L
323
285
38
L
T
R
R
T
137
198
L
61
L
T
R
0
0
0
MidBlock Vol
11
MidBlock Vol
-22
Total Int. Vol. = 700

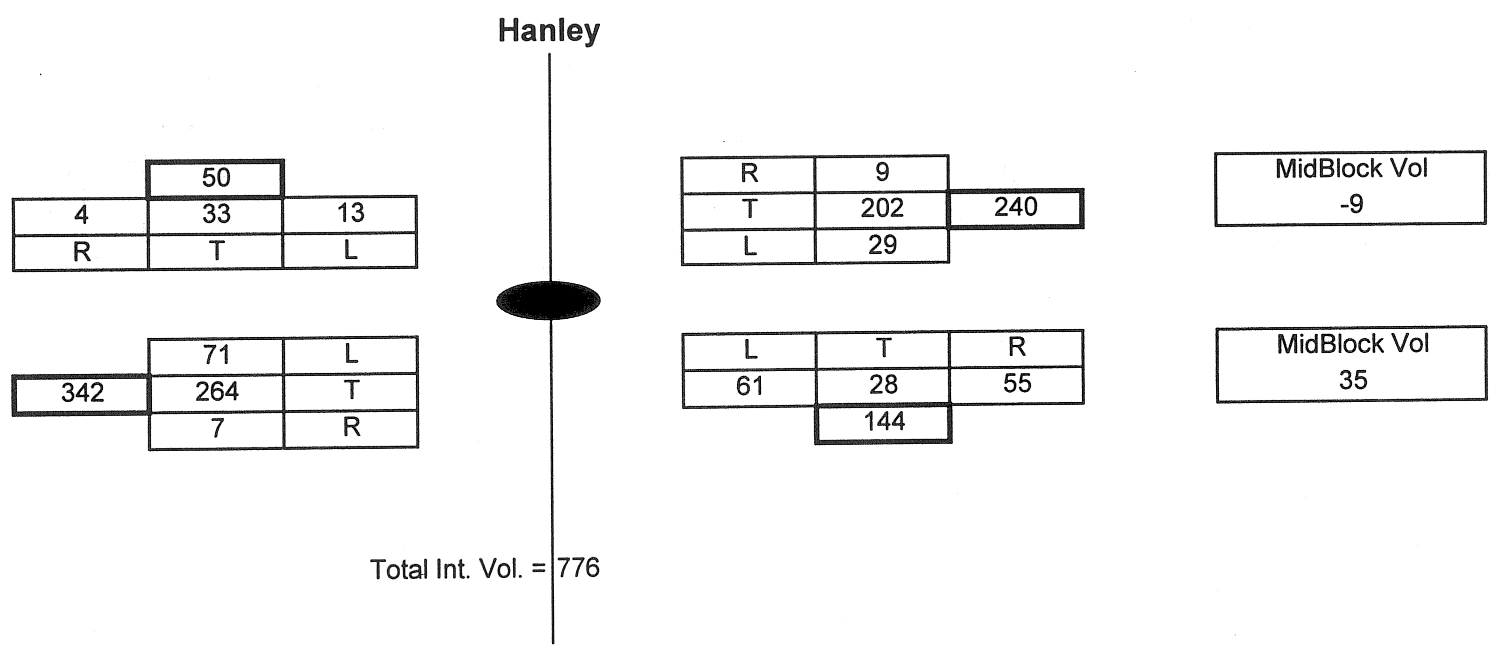

WB Ramp
R
T
L
70
L
395
325
T
R
R
115
T
170
285
L
L
T
R
17
61
78
MidBlock Vol
-40
MidBlock Vol
48
Total Int. Vol. = 758

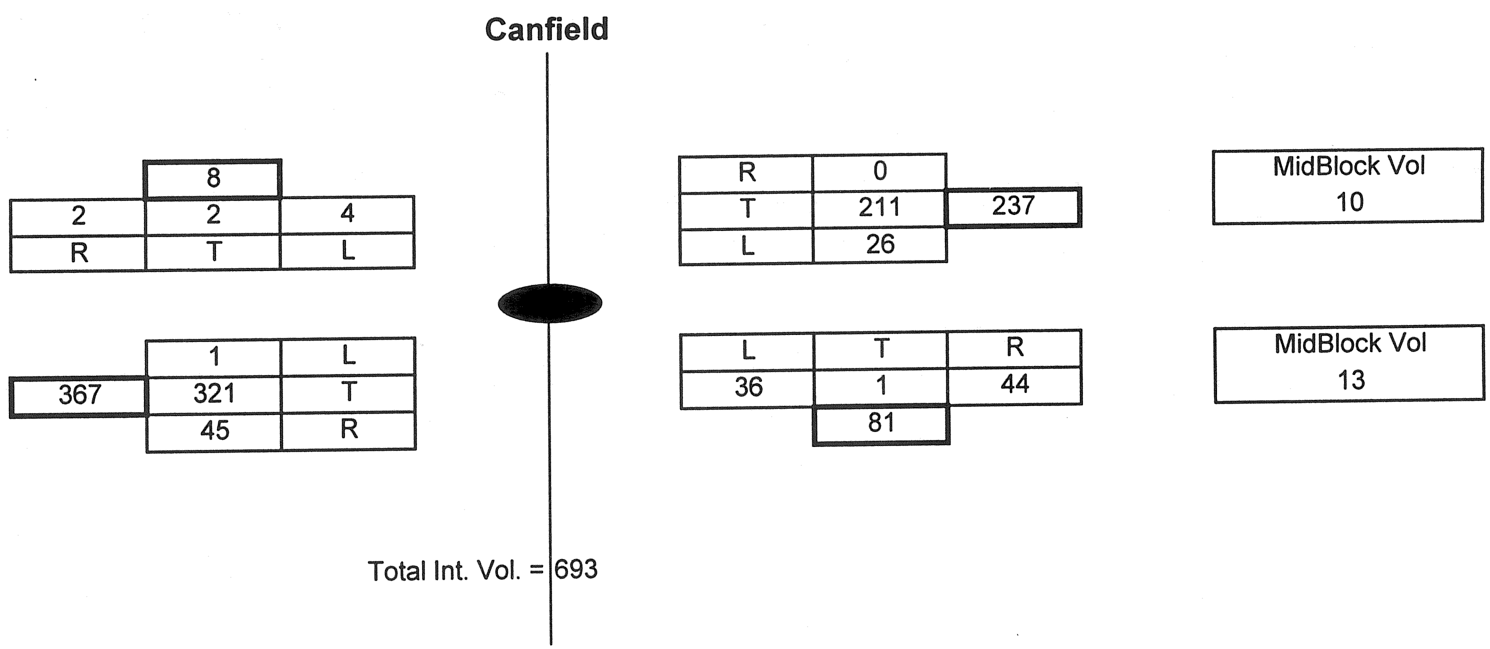

Appleway
157
47
81
29
R
T
L
52
L
434
310
T
72
R
R
36
T
220
299
L
43
L
T
R
58
105
59
222
MidBlock Vol
-55
MidBlock Vol
-30
Total Int. Vol. = 1112

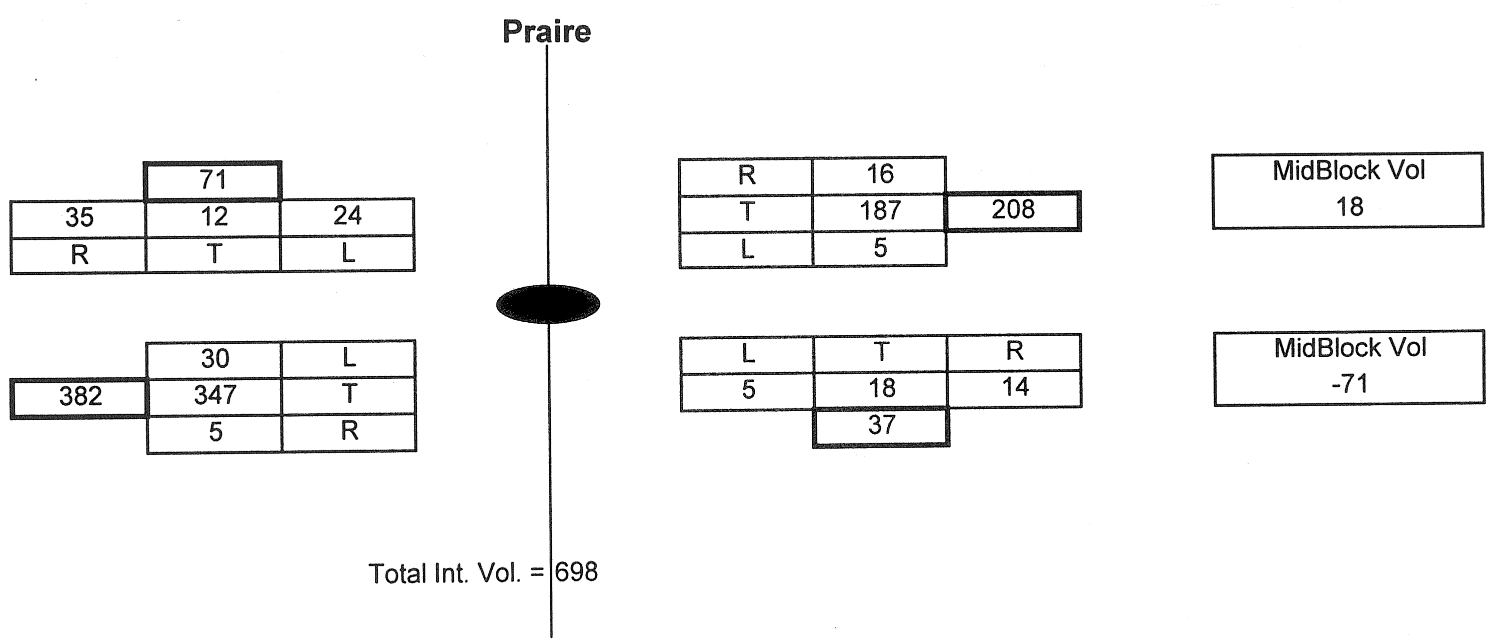

Neider
66
12
17
37
R
T
L
10
L
368
326
T
32
R
R
46
T
324
404
L
34
L
T
R
18
17
50
85
MidBlock Vol
98
MidBlock Vol
-54
Total Int. Vol. = 923

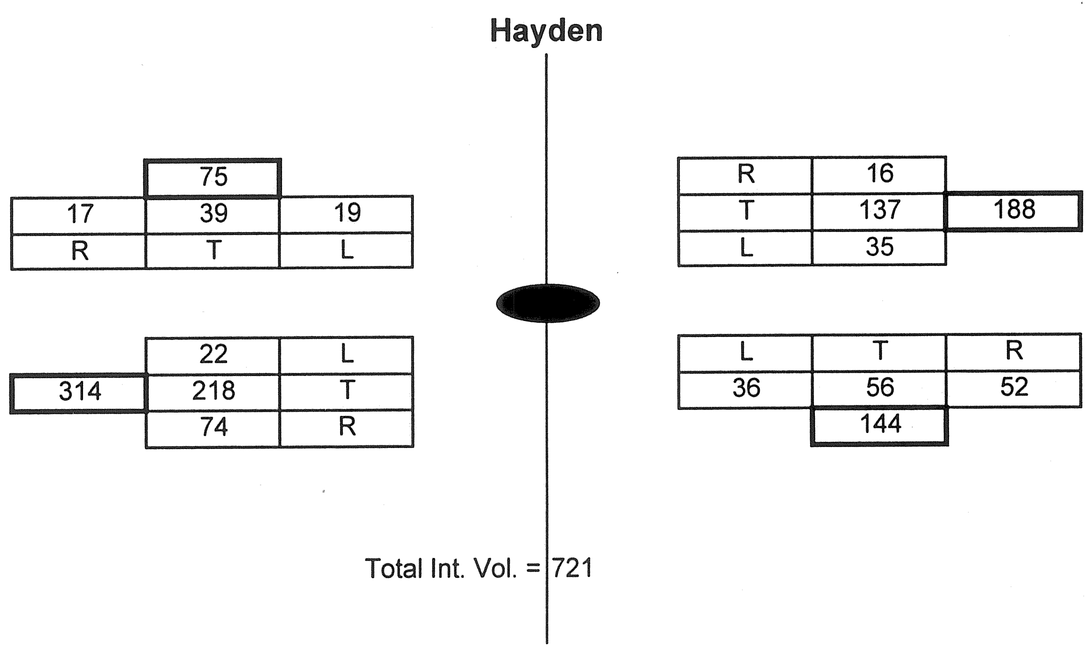

Kathleen
96
30 40 26
R T L
33 L
359 292 T
34 R
R 24
T 256 301
L 21
L T R
20 42 54
116
MidBlock Vol
31
MidBlock Vol
-14
Total Int. Vol. = 872


Dalton
75
27
29
19
R
T
L
43
L
358
301
T
14
R
R
22
T
221
257
L
14
L
T
R
22
25
19
66
MidBlock Vol
-10
MidBlock Vol
3
Total Int. Vol. = 756


Hanley
50
4
33
13
R
T
L
71
L
342
264
T
7
R
R
9
T
202
240
L
29
L
T
R
61
28
55
144
MidBlock Vol
-9
MidBlock Vol
35
Total Int. Vol. = 776


Canfield
8
2
2
4
R
T
L
1
L
367
321
T
45
R
R
0
T
211
237
L
26
L
T
R
36
1
44
81
MidBlock Vol
10
MidBlock Vol
13
Total Int. Vol. = 693


Praire
71
35
12
24
R
T
L
30
L
382
347
T
5
R
R
16
T
187
208
L
5
L
T
R
5
18
14
37
MidBlock Vol
18
MidBlock Vol
-71
Total Int. Vol. = 698


Hayden
75
17
39
19
R
T
L
22
L
314
218
T
74
R
R
16
T
137
188
L
35
L
T
R
36
56
52
144
Total Int. Vol. = 721

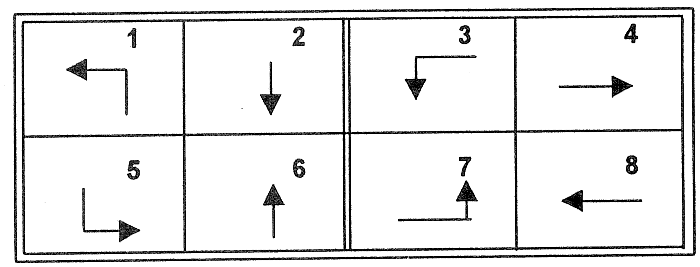
D. Most Frequently Occurring Phasing Sequence

Most Frequently Occuring Phasing Sequence

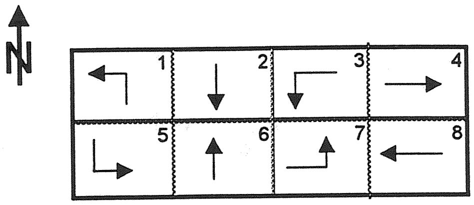
Day:	Friday	
Time Period:	PM	
Date:		10/25/96
Weather:	Rainy	

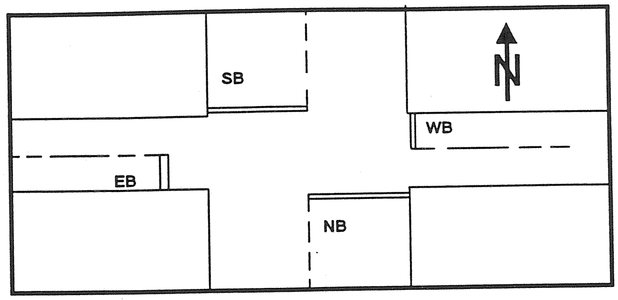
U.S. 95 & Cross Street	Phasing Sequence
Hayden	26,37,48,15
Praire	26,37,48,16
Canfield	26,37,47,25
Hanley	25,26,3,4
Dalton	26,37,47,48,16
Kathleen	37,48,15,25,26
Neider	26,37,48,15
Appleway	15,26,8,7
WB Ramp	16,4
EB Ramp	26,25,4
Ironwood	26,37,38,48,15

Phasing Diagram



E. LM System Signal Timing





Signal Timing from the LM System Software
Cycle 2 Length = 120 seconds for Split 1

Ironwood	Phase 26	Phase 37	Phase 48	Phase 15	
Y	3.6	3.1	3.1	3.6	13.4
AR	1.9	2	2	1.9	7.8
Y+AR	*5.5*	*5.1*	*5.1*	*5.5*	
Sequence	37	61	93	114	
Green	*37*	*18.5*	*27.4*	*15.9*	98.8
Cycle Length					120

E. Ramp	Phase 6	Phase 25	Phase 4	Phase 1378	
Y	3.6	3.6	3.1		10.3
AR	1.1	1.1	1.6	0	3.8
Y+AR	*4.7*	*4.7*	*4.7*	*0*	
Sequence	41	72	115	120	
Green	41	26.3	38.6	0	105.9
Cycle Length					120

W. Ramp	Phase 2	Phase 16	Phase 8	Phase 3578	
Y	3.6	3.1	3.1		9.8
AR	1.1	1.6	1.6	0	4.3
Y+AR	*4.7*	*4.7*	*4.7*	*0*	
Sequence	41	72	115	120	
Green	41	26.3	38.6	0	105.9
Cycle Length					120

Appleway	Phase 15	Phase 26	Phase 38	Phase 47	Phase	
Y	3.6	3.6	3.6	3.6	0	14.4
AR	1	1	1.5	1.5	0	5
Y+AR	*4.6*	*4.6*	*5.1*	*5.1*	|	
Sequence	29	53	84	114	120	
Green	29	19.4	26.8	25.4		100.6
Cycle Length						120

Neider	Phase 26	Phase 37	Phase 48	Phase 15	
Y	4	3.2	3.2	3.2	13.6
AR	1.9	3.3	3.8	2.6	11.6
Y+AR	*5.9*	*6.5*	*7*	*5.8*	
Sequence	48	72	92	113	
Green	48	18.1	14.7	14	94.8
Cycle Length					120

Kathleen	Phase 26	Phase 37	Phase 48	Phase 15	
Y	4	3.2	3.2	3.2	13.6
AR	2	3.3	3.7	2.6	11.6
Y+AR	*6*	*6.5*	*6.9*	*5.8*	
Sequence	50	68	85	113	
Green	50	12	11.7	21.1	94.8
Cycle Length					120

Dalton	Phase 26	Phase 37	Phase 48	Phase 15	
Y	4	3.2	3.2	3.2	13.6
AR	1.6	3.2	3.6	1.9	10.3
Y+AR	***5.6***	***6.4***	***6.8***	***5.1***	
Sequence	49	72	89	113	
Green	49	17.4	12.5	17.2	96.1
Cycle Length					120

Hanley	Phase 26	Phase 38	Phase 47	Phase 15	Phase	
Y	4	3.2	3.2	3.2		13.6
AR	1.9	3.6	3.6	2.4	0	11.5
Y+AR	***5.9***	***6.8***	***6.8***	***5.6***		
Sequence	49	72	94	115	120	
Green	49	16.5	15.2	14.2		94.9
Cycle Length						120

Canfield	Phase 26	Phase 37	Phase 48	Phase 15	
Y	4	3.2	3.2	3.2	13.6
AR	1.6	3.2	3.6	1.9	10.3
Y+AR	***5.6***	***6.4***	***6.8***	***5.1***	
Sequence	39	62	88	114	
Green	39	17.4	20.5	19.2	96.1
Cycle Length					120

Prairie	Phase 26	Phase 37	Phase 48	Phase 15	
Y	4	3.2	3.2	3.2	13.6
AR	1.8	3.5	4	2.4	11.7
Y+AR	***5.8***	***6.7***	***7.2***	***5.6***	
Sequence	41	64	94	115	
Green	41	16.6	23.3	13.8	94.7
Cycle Length					120

Hayden	Phase 26	Phase 37	Phase 48	Phase 15	
Y	4	3.2	3.2	3.2	13.6
AR	1.8	3.1	3.5	2.3	10.7
Y+AR	***5.8***	***6.3***	***6.7***	***5.5***	
Sequence	37	67	90	113	
Green	37	24.2	18.2	16.3	95.7
Cycle Length					120

F. Offsets

OFFSETS DETERMINATION FOR BOTH NORTH AND SOUTH BOUNDS

DATE:	10/25/96	**DAY:**	Friday
WEATHER:	Rainy	**Observer:**	Sanjeev Kumar Tandle
Time - Period analyzed:	PM	**Time:**	5:00 - 5:15

North Bound: Ironwood to Hayden
South Bound: Hayden to Ironwood

Hayden & Praire

Cycle #	Offsets (sec)	
	NB	SB
1	0	0
2	-50	50
3	-7	7
4	5	-5
5	-45	-67
6	-32	32

Praire & Canfield

Cycle #	Offsets (sec)	
	NB	SB
1	0	0
2	80	-80
3	55	-55
4	54	-54
5	132	-132
6	127	-127

Canfield & Hanley

Cycle #	Offsets (sec)	
	NB	SB
1	0	0
2	-13	12
3	-30	30
4	-26	25
5	-44	44
6	-53	53
7	-66	65
8	-95	94

Hanley & Dalton

Cycle #	Offsets (sec)	
	NB	SB
1	-106	-
2	-14	15
3	-14	14
4	-30	31
5	-39	39
6	-42	42
7	-58	59
8	-58	59

Dalton & Kathleen

Cycle #	Offsets (sec)	
	NB	SB
1	14	-
2	-95	116
3	-96	115
4	-96	113
5	-97	114
6	-112	98
7	-93	93

Kathleen & Neider

Cycle #	Offsets (sec)	
	NB	SB
1	-	-8
2	92	-113
3	92	-111
4	99	-116
5	100	-117
6	116	-102
7	94	-94

Neider & Appleway

Cycle #	Offsets (sec)	
	NB	SB
1	-	-66
2	-30	30
3	-30	30
4	-38	38
5	-39	39
6	-39	39
7	-39	39

Appleway & West Ramp

Cycle #	Offsets (sec)	
	NB	SB
1	34	-
2	72	-72
3	80	-80
4	156	-156
5	198	-198
6	217	-163
7	231	-214

West Ramp & East Ramp

Cycle #	Offsets (sec)	
	NB	SB
1	0	-
2	0	0
3	5	-5
4	-36	36
5	-73	-4
6	-86	9
7	-58	-19

East Ramp & Ironwood

Cycle #	Offsets (sec)	
	NB	SB
1	0	0
2	-34	34
3	-78	78
4	-69	69
5	-73	73
6	-71	71
7	-106	106
8	-95	95

G. LOS from HCS

Existing Level of Service data sheet

Date: 10/24/96 Day: Thursday

Period: PM Tool used: HCS

Cross street	Existing Level of Service																
	North Bound				South Bound				East Bound				West Bound				Intersection
	Left	Through	Right	Total	Left	Through	Right	Total	Left	Through	Right	Total	Left	Through	Right	Total	LOS
Ironwood	D	C	C	C	D	C	C	C	D	D	D	D	D	D	D	D	D
East Ramp	-	C	C	C	D	D	-	D	D	-	C	D	-	-	-	-	D
West Ramp	D	*	-	*	-	C	C	C	-	-	-	-	C	-	C	C	*
Appleway	C	*	F	*	C	*	D	*	C	D	D	D	D	D	D	D	*
Neider	D	C	B	C	*	C	B	*	D	D	D	D	D	D	D	D	*
Kathleen	D	C	B	C	D	C	B	C	D	D	D	D	D	D	D	D	C
Dalton	D	C	B	C	D	C	B	C	D	D	D	D	D	D	D	D	C
Hanley	F	C	B	D	D	C	B	C	D	D	D	D	D	D	E	D	D
Canfield	D	E	C	D	D	D	C	D	D	C	C	D	D	D	D	D	D
Prairie	D	D	C	D	D	C	C	C	D	C	D	D	D	D	D	D	D
Hayden	D	C	C	C	D	C	C	C	C	E	E	D	C	*	*	*	*

* Delay is meaningless when v/c ratio is greater than 1.2

H. SIDRA LOS and Delay

Existing Level of Service

Date: 10/25/96 **Day:** Friday

Period: 5:00 - 5:15 PM **Tool used:** SIDRA
Cycle length,C: 120 sec **Version:** 5

Intersection	North Bound				South Bound				East Bound				West Bound				Intersection
	Left	Through	Right	Total	Left	Through	Right	Total	Left	Through	Right	Total	Left	Through	Right	Total	LOS
Ironwood	D	D	D	D	E	D	A	D	E	D	D	D	D	D	A	C	D
East Ramp	-	D	A	D	E	E	-	E	E	E	A	D	-	-	-	-	D
West Ramp	B	D	-	D	-	D	D	D	-	-	-	-	E	E	B	C	D
Appleway	F	F	A	F	F	F	A	F	D	D	D	D	C	C	B	C	F
Neider	E	C	A	C	E	C	A	C	E	C	B	D	E	E	B	C	C
Kathleen	E	C	A	C	E	C	A	C	E	C	A	C	E	E	B	C	C
Dalton	F	F	F	F	E	F	A	F	E	B	B	C	F	F	B	D	F
Hanley	F	E	A	E	D	D	A	D	D	E	E	D	F	F	B	D	E
Canfield	E	C	A	C	E	C	A	C	E	C	C	D	E	E	B	D	C
Prairie	E	C	A	C	E	B	A	B	E	D	A	C	E	E	B	C	C
Hayden	E	D	A	D	E	D	A	D	E	C	C	D	E	E	D	D	D

Optimized Level of Service

Date: 10/25/96 **Day:** Friday

Period: 5:00 - 5:15 PM **Tool used:** SIDRA
Version: 5

Intersection	North Bound				South Bound				East Bound				West Bound				Intersection
	Left	Through	Right	Total	Left	Through	Right	Total	Left	Through	Right	Total	Left	Through	Right	Total	LOS
Ironwood	D	D	D	D	D	D	A	D	D	D	D	D	D	D	B	C	D
East Ramp	-	E	A	D	D	E	-	E	E	E	A	D	-	-	-	-	D
West Ramp	B	D	-	D	-	D	E	D	-	-	-	-	F	F	B	C	D
Appleway	F	E	A	E	F	F	A	E	D	D	C	D	D	D	D	D	E
Neider	E	C	A	C	E	C	A	C	E	D	A	D	E	E	B	C	C
Kathleen	E	C	A	C	E	C	A	C	E	C	A	C	E	D	B	C	C
Dalton	E	D	A	D	D	C	A	C	E	D	D	D	E	D	D	D	D
Hanley	E	D	A	E	E	D	A	D	E	E	E	E	E	E	A	D	D
Canfield	E	C	A	C	E	C	A	C	E	C	C	D	E	B	B	D	C
Prairie	F	C	A	C	E	B	A	B	F	E	A	D	E	D	B	C	C
Hayden	D	D	A	D	E	C	A	D	D	C	C	C	E	D	D	D	D

Existing Average Delay

Date: 10/25/96 **Day:** Friday

Period: 5:00 - 5:15 PM **Tool used:** SIDRA
Cycle length,C: 120 sec **Version:** 5

Intersection	North Bound				South Bound				East Bound				West Bound				Intersection
	Left	Through	Right	Total	Left	Through	Right	Total	Left	Through	Right	Total	Left	Through	Right	Total	Delay
Ironwood	53.7	50.7	52.9	51.3	61	42	7.5	44.7	61.5	45.3	44.2	50.5	52.7	43.4	14.1	28.2	44.9
East Ramp	-	49.7	7.7	44.7	68.8	71.2	-	70.5	66.2	66.2	12	52.2	-	-	-	-	53.9
West Ramp	21.2	40.3	-	36.9	-	37.8	58.8	46.3	-	-	-	-	75.4	75.4	21.5	34.5	40.2
Appleway	374.2	205.3	9.2	193	246	242.9	10.3	215.3	52.4	45.9	41.2	45.7	38.6	28.8	18.5	28.6	141.1
Neider	65.3	31.7	9.1	30.7	77.5	31.1	9	32.5	72.5	31.5	12.8	51.1	64.1	44.2	22.5	35.6	33.7
Kathleen	70.9	24.6	9.7	27.4	65.4	23.1	8.5	24.9	71.5	31.4	8.6	35	68	41.4	16	34.1	28.8
Dalton	312.2	318.5	9.7	305.7	71.2	132.4	9.8	118.6	84.1	17.9	17.9	34.7	114.7	16.9	16.9	49.5	182.7
Hanley	94.1	65.8	9	70.5	57.4	39	9.2	40.1	57.6	54	54	55	93.1	44.6	16.2	54.3	56.2
Canfield	67.1	27.7	9.5	25.7	76.5	20.5	10.5	26.5	63.2	26.6	26.6	43.9	77.7	18.8	18.8	44.5	28.9
Prairie	80.8	25.2	10.4	29.4	67.3	16.6	10.4	17.3	83.8	43.6	10.4	40.9	69.9	34.4	15.6	32.3	27.8
Hayden	69.1	47	10.8	40	76.6	38.9	10.8	43.5	68.4	29.3	29.3	39.2	78	48.2	48.2	55.6	43.2

Minimized Average Delay

Date: 10/25/96 **Day:** Friday

Period: 5:00 - 5:15 PM **Tool used:** SIDRA
Version: 5

Intersection	North Bound				South Bound				East Bound				West Bound				Intersection	Optimised cycle length
	Left	Through	Right	Total	Left	Through	Right	Total	Left	Through	Right	Total	Left	Through	Right	Total	Delay	
Ironwood	47.7	49.4	51.3	49.3	57.3	37.4	7.5	40.9	58.2	36.5	35.5	43.5	46.8	35	14.8	25.2	40.8	100
East Ramp	-	56	7.7	50.3	58.8	58.3	-	58.5	66.4	66.4	11.7	52.2	-	-	-	-	53.1	100
West Ramp	22.9	36.6	-	34.2	-	41.7	59	48.7	-	-	-	-	86.6	86.6	21.9	37.5	39.9	140
Appleway	106.9	67.7	10.7	63	69.6	86.9	13	75.5	46.8	40.8	38.5	41.2	50.4	53.1	47.4	50.9	60.1	110
Neider	72.8	26	9.1	25.8	81.4	25.9	9	28.6	78.9	36	11.2	55.5	71.8	55.1	22.1	39.2	31.1	140
Kathleen	70.9	24.6	9.7	27.4	65.4	23.1	8.5	24.9	71.5	31.4	8.6	35	68	41.4	16	34.1	28.8	120
Dalton	70.7	34	9.7	37.5	55.4	23.3	9.8	23.9	65.3	54.6	54.6	57.3	69.7	38.1	38.1	48.6	35.6	100*
Hanley	84.1	51.2	9	57.2	66	41.2	9.5	43	66.7	72.9	72.9	71.3	86.2	61.5	15.1	54.2	53	150
Canfield	71.3	26.4	9.5	24.6	80.4	20.9	10.5	27.2	66.3	29.2	29.2	46.8	77.4	18.9	18.9	44.5	28.8	130
Prairie	88.1	20.1	10.4	25.4	78.8	15.1	10.4	16.3	88.4	59.9	10.4	45.2	81.2	43	13.6	37.3	26.6	150
Hayden	55.6	46.7	10.8	38.9	63.4	32	10.8	36.1	55	23.8	23.8	31.7	65.1	55	55	57.5	40.3	90*

* indicates that sidra could not able to solve the problem may be due to high congestion

I-I. EXISTING, OPTIMIZED, and PROPOSED LOS and Signal Timing

OVERALL INTERSECTION
EXISTING LOS = D
OPTIMIZED LOS = C
PROPOSED LOS D

Southbound approach

Prop.	LOS	C	C	E
Optim.	LOS	B	B	D
Exist.	LOS	B+	C	D
	satflow	1477	3226	1593
	width	14	24	12
	# lane	1	2	1
	volume	64	548	140

Westbound approach

volume	lanes	width	satflow	Exist. LOS	Optim. LOS	Prop. LOS
208	~	~	~	~	~	~
224	1	14	1606	F	C	D
144	1	12	1593	D+	C+	D

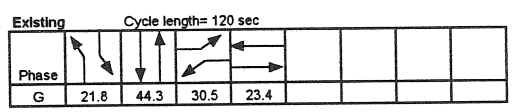
HAYDEN

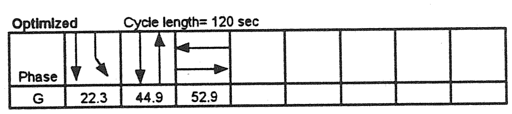
Eastbound approach

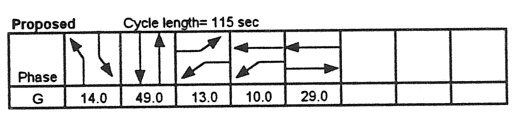
Prop. LOS	Optim. LOS	Exist. LOS	satfolw	width	lanes	volume
E	D+	D+	1593	12	1	76
D	C+	E+	1649	14	1	156
~	~	~	~	~	~	68

Northbound approach

	volume	88	872	296
	lanes	1	2	1
	width	12	24	14
	satflow	1593	3226	1477
Exist	LOS	D	D+	B+
Optim.	LOS	D+	D+	D+
Prop.	LOS	D	D	C







N

Legend
G = Stage Length (sec)

Diagram I-I-1: Hayden Exsiting/Optimized Data

OVERALL INTERSECTION
EXISTING LOS = **D**
OPTIMIZED LOS = **B**
PROPOSED LOS **B**

Prop.	LOS	B	B	D
Optim.	LOS	B+	B+	B+
Exist.	LOS	B+	D+	D
	satflow	1408	3173	1540
	width	12.5	23	11
	# lane	1	2	1
	volume	64	748	20

volume	lanes	width	satflow	Exist. LOS	Optim. LOS	Prop. LOS
56	~	~	~	~	~	~
72	2	23.5	3018	D+	C	D
20	1	11.5	1566	D+	C+	D

PRAIRIE

Prop. LOS	Optim. LOS	Exist. LOS	satfolw	width	lanes	volume
E	C	D	1486	10	1	96
D	C	D+	2851	23	2	48
~	~	~	~	~	~	140

	volume	120	1388	20
	lanes	1	2	1
	width	11.5	23	12.5
	satflow	1566	3173	1408
Exist.	LOS	D	E+	B+
Optim.	LOS	B	B	B+
Prop.	LOS	E	B	B

Existing Cycle length= 120 sec

Phase								
G	18.8	46.8	23.9	30.5				

Optimized Cycle length= 120 sec

Phase								
G	75.2	44.8						

Proposed Cycle length= 115 sec

Phase								
G	12.0	70.0	13.0	20.0				

N

Legend
G = Stage Length (sec)

Diagram I-I-2: Prairie Exsiting/Optimized Data

Prop.	LOS	C	C	D
Optim.	LOS	C	D+	D
Exist.	LOS	C+	C	D
	satflow	1524	3146	1566
	width	15	22.5	11.5
	# lane	1	2	1
	volume	36	808	116

OVERALL INTERSECTION
EXISTING LOS = D
OPTIMIZED LOS = D+
PROPOSED LOS C

volume	lanes	width	satflow	Exist. LOS	Optim. LOS	Prop. LOS
220	~	~	~	~	~	~
112	2	22	2830	D	C	D
244	1	10.5	1593	F	D+	D

HANLEY

Prop. LOS	Optim. LOS	Exist. LOS	satfolw	width	lanes	volume
D	C	D	1593	10.5	1	52
E	C	D	1513	11	1	132
~	~	~	~	~	~	16

	volume	284	1056	28
	lanes	1	2	1
	width	13	23	13.5
	satflow	1646	3174	1454
Exist.	LOS	F	D+	B+
Optim.	LOS	D	D+	C+
Prop.	LOS	D	B	B

Existing Cycle length= 120 sec

Phase								
G	15.8	54.9	23.9	22.0				

Optimized Cycle length= 120 sec

Phase								
G	20.2	11.9	42.5	45.4				

Proposed Cycle length= 115 sec

Phase								
G	15.0	17.0	45.0	16.0	22.0			

N

Legend
G = Stage Length (sec)

Diagram I-I-4: Hanley Exsiting/Optimized Data

OVERALL INTERSECTION
EXISTING LOS = D
OPTIMIZED LOS = B
PROPOSED LOS C

Prop.	LOS	B	B	D
Optim.	LOS	A	A	D
Exist.	LOS	B+	D+	D
	satflow	1385	3226	1593
	width	12	24	12
	# lane	1	2	1
	volume	1	844	104

volume	lanes	width	satflow	Exist. LOS	Optim. LOS	Prop. LOS
176	~	~	~	~	~	~
4	1	13	1437	D	D	D
144	1	12	1593	D	D+	D

CANFIELD

Prop. LOS	Optim. LOS	Exist. LOS	satfolw	width	lanes	volume
D	D+	D+	1486	10	1	16
D	D+	D+	1408	10	1	8
~	~	~	~	~	~	8

	volume	4	1284	180
	lanes	1	2	1
	width	12	24	12
	satflow	1593	3226	1385
Exist.	LOS	D+	E+	B+
Optim.	LOS	B+	C+	B
Prop.	LOS	D	C	B

Existing Cycle length= 120 sec

Phase								
G	24.3	45.5	23.8	26.4				

Optimized Cycle length= 120 sec

Phase								
G	21.5	71.6	28.9					

Proposed Cycle length= 115 sec

Phase								
G	9.0	10.0	59.0	12.0	15.0	10.0		

N

Legend
G = Stage Length (sec)

Diagram I-I-3: Canfield Exsiting/Optimized Data

OVERALL INTERSECTION
EXISTING LOS = D
OPTIMIZED LOS = C
PROPOSED LOS B

Prop.	LOS	B	B	D
Optim.	LOS	B	C+	D
Exist.	LOS	B+	C	D
	satflow	1431	3200	1593
	width	13	23.5	12
	# lane	1	2	1
	volume	88	884	56

volume	lanes	width	satflow	Exist. LOS	Optim. LOS	Prop. LOS
76	~	~	~	~	~	~
100	1	12	1523	F	D+	D
88	1	12	1593	D	D+	D

DALTON

Prop. LOS	Optim. LOS	Exist. LOS	satfolw	width	lanes	volume
D	D+	D	1593	12	1	76
D	D+	F	1511	12	1	116
~	~	~	~	~	~	108

	volume	172	1204	56
	lanes	1	2	1
	width	13	23	13
	satflow	1646	3173	1431
Exist.	LOS	D	D+	B+
Optim.	LOS	D	C+	B
Prop.	LOS	D	B	B

Existing Cycle length= 120 sec

Phase								
G	22.3	56.5	23.8	17.4				

Optimized Cycle length= 120 sec

Phase								
G	20.0	6.3	59.4	34.3				

Proposed Cycle length= 115 sec

Phase								
G	11.0	55.0	17.0	12.0	20.0			

N

Legend
G = Stage Length (sec)

Diagram I-I-5: Dalton Exsiting/Optimized Data

OVERALL INTERSECTION	
EXISTING LOS =	D
OPTIMIZED LOS =	C
PROPOSED LOS	C

Prop.	LOS	B	C	D
Optim.	LOS	B	C+	D
Exist.	LOS	B+	C	D+
	satflow	1408	3137	1593
	width	12.5	23	12
	# lane	1	2	1
	volume	96	1024	84

volume	lanes	width	satflow	Exist. LOS	Optim. LOS	Prop. LOS
216	~	~	~	~	~	~
168	2	22	2887	F	D+	D
80	1	12	1593	D	D+	D

KATHLEEN

Prop. LOS	Optim. LOS	Exist. LOS	satfolw	width	lanes	volume
E	D+	D	1646	13	1	104
D	D+	E+	2996	23	2	160
~	~	~	~	~	~	120

	volume	132	1168	136
	lanes	1	2	1
	width	12	23.5	14
	satflow	1699	3200	1385
Exist.	LOS	D+	C	B+
Optim.	LOS	D	C	B
Prop.	LOS	D	B	B

Existing Cycle length= 120 sec

Phase								
G	26.9	57.2	18.5	17.4				

Optimized Cycle length= 120 sec

Phase								
G	21.9	60.4	37.7					

Proposed Cycle length= 115 sec

Phase								
G	23.0	41.0	18.0	14.0	19.0			

N

Legend

G = Stage Length (sec)

Diagram I-I-6: Kathleen Exsiting/Optimized Data

Prop.	LOS	B	B	D
Optim.	LOS	B+	C+	B+
Exist.	LOS	B+	B	D
	satflow	1385	3226	1593
	width	12	24	12
	# lane	1	2	1
	volume	80	1185	40

OVERALL INTERSECTION
EXISTING LOS = **D**
OPTIMIZED LOS = **C+**
PROPOSED LOS **B**

volume	lanes	width	satflow	Exist. LOS	Optim. LOS	Prop. LOS
60	~	~	~	~	~	~
10	1	12	1434	D	D+	D
40	1	12	1593	D	D+	D

BOSANKO

Prop. LOS	Optim. LOS	Exist. LOS	satfolw	width	lanes	volume
D	D+	E+	1593	12	1	100
D	D+	E+	1415	12	1	10
~	~	~	~	~	~	80

	volume	100	1205	60
	lanes	1	2	1
	width	12	24	12
	satflow	1699	3200	1385
Exist.	LOS	E+	B+	A
Optim.	LOS	D	B+	A
Prop.	LOS	D	B	B

Existing Cycle length= 120 sec

Phase								
G	16.0	72.0	16.0	16.0				

Optimized Cycle length= 120 sec

Phase								
G	23.7	68.4	27.9					

Proposed Cycle length= 115 sec

Phase								
G	14.0	69.0	14.0	18.0				

N

Legend
G = Stage Length (sec)

Diagram I-I-7: Bosanko Exsiting/Optimized Data

Prop.	LOS	B	B	D
Optim.	LOS	B	C+	D
Exist.	LOS	B+	E+	D
	satflow	1431	3200	1593
	width	13	23.5	12
	# lane	1	2	1
	volume	184	1296	136

OVERALL INTERSECTION
EXISTING LOS = D
OPTIMIZED LOS = C
PROPOSED LOS C

volume	lanes	width	satflow	Exist. LOS	Optim. LOS	Prop. LOS
200	~	~	~	~	~	~
68	2	22.5	2826	D	D+	D
72	1	11.5	1566	D+	C	D

NEIDER

Prop. LOS	Optim. LOS	Exist. LOS	satfolw	width	lanes	volume
E	D	D	1513	10.5	1	148
D	C	D	3028	23.5	2	68
~	~	~	~	~	~	48

	volume	40	1304	128
	lanes	1	2	1
	width	12	24	13
	satflow	1593	3200	1431
Exist.	LOS	D	E+	B+
Optim.	LOS	D	C	B
Prop.	LOS	D	B	B

Existing Cycle length= 120 sec

Phase								
G	19.8	53.9	24.6	20.5				

Optimized Cycle length= 120 sec

Phase								
G	13.8	7.4	61.5	37.3				

Proposed Cycle length= 115 sec

Phase								
G	11.0	8.0	61.0	14.0	21.0			

N

Legend
G = Stage Length (sec)

Diagram I-I-8: Neider Exsiting/Optimized Data

OVERALL INTERSECTION

EXISTING LOS = **D**
OPTIMIZED LOS = **D**
PROPOSED LOS = **D**

Southbound approach

Prop.	LOS	C	C	D
Optim.	LOS	C	D	D
Exist.	LOS	D	F	D+
	satflow	1385	3226	1593
	width	12	24	12
	# lane	1	2	1
	volume	144	880	172

Westbound approach

volume	lanes	width	satflow	Exist. LOS	Optim. LOS	Prop. LOS
236	~	~	~	~	~	~
420	2	23	3026	E+	D	D
232	1	10	1486	D	D	D

APPLEWAY

Eastbound approach

Prop. LOS	Optim. LOS	Exist. LOS	satfolw	width	lanes	volume
D	D	D+	1486	10	1	116
D	D	D	3023	23	2	324
~	~	~	~	~	~	188

Northbound approach

	volume	208	1240	288
	lanes	1	3	1
	width	12	36	11
	satflow	1593	4840	1339
Exist.	LOS	D+	E	C+
Optim.	LOS	D	D	B
Prop.	LOS	E	C	C

Existing Cycle length= 120 sec

Phase								
G	33.6	24.9	31.5	30.0				

Optimized Cycle length= 120 sec

Phase								
G	24.7	39.9	20.1	7.5	27.8			

Proposed Cycle length= 115 sec

Phase								
G	21.0	21.0	22.0	22.0	29.0			

N

Legend

G = Stage Length (sec)

Diagram I-I-9: Appleway Exsiting/Optimized Data

OVERALL INTERSECTION
EXISTING LOS = **C**
OPTIMIZED LOS = **C+**
PROPOSED LOS = **B**

Prop.	LOS	B	B	~
Optim.	LOS	D+	C	~
Exist.	LOS	D	D+	~
	satflow	1378	3173	~
	width	11	23	~
	# lane	1	2	~
	volume	460	680	~

volume	lanes	width	satflow	Exist. LOS	Optim. LOS	Prop. LOS
244	~	~	~	~	~	~
0	1	11	1378	C	D+	E
68	1	10	1516	C	D+	D

WB RAMP

Prop. LOS	Optim. LOS	Exist. LOS	satfolw	width	lanes	volume
~	~	~	~	~	~	~
~	~	~	~	~	~	~
~	~	~	~	~	~	~

	volume	280	1300	~
	lanes	1	2	~
	width	13	24	~
	satflow	1646	3226	~
Exist.	LOS	D	B	~
Optim.	LOS	D+	B+	~
Prop.	LOS	D	A	~

Existing Cycle length= 120 sec

Phase							
G	45.7	31.0	43.0				

Optimized Cycle length= 120 sec

Phase							
G	52.1	34.4	33.5				

Proposed Cycle length= 115 sec

Phase							
G	30.0	65.0	20.0				

N

Legend
G = Stage Length (sec)

Diagram I-I-10: WB Ramp Exsiting/Optimized Data

OVERALL INTERSECTION	
EXISTING LOS =	D
OPTIMIZED LOS =	D
PROPOSED LOS =	C

Prop.	LOS	~	B	D
Optim.	LOS	~	B	D
Exist.	LOS	~	B+	D
	satflow	~	3226	1699
	width	~	24	14
	# lane	~	2	1
	volume	~	548	244

volume	lanes	width	satflow	Exist. LOS	Optim. LOS	Prop. LOS
~	~	~	~	~	~	~
~	~	~	~	~	~	~
~	~	~	~	~	~	~

EB RAMP

Prop. LOS	Optim. LOS	Exist. LOS	satfolw	width	lanes	volume
D	D	E	1399	11	1	528
~	~	~	~	~	~	~
C	C+	C	1339	11	1	188

	volume	~	1140	152
	lanes	~	2	1
	width	~	24	11
	satflow	~	3226	1366
Exist.	LOS	~	E+	C
Optim.	LOS	~	E	D+
Prop.	LOS	~	C	B

Existing Cycle length= 120 sec

Phase								
G	45.7	31.0	43.0					

Optimized Cycle length= 120 sec

Phase								
G	36.0	28.4	55.6					

Proposed Cycle length= 115 sec

Phase								
G	48.0	28.0	39.0					

N

Legend

G = Stage Length (sec)

Diagram I-I-11: EB Ramp Exsiting/Optimized Data

Prop.	LOS	C	C	E
Optim.	LOS	D+	D	D
Exist.	LOS	B	C	F
	satflow	1293	3280	1699
	width	10	25	14
	# lane	1	2	1
	volume	88	416	268

OVERALL INTERSECTION

EXISTING LOS = D

OPTIMIZED = D

PROPOSED LOS = D

volume	lanes	width	satflow	Exist. LOS	Optim. LOS	Prop. LOS
348	~	~	~	~	~	~
180	2	25	2990	D	D	D
88	1	14	1699	D	D	D

IRONWOOD

Prop. LOS	Optim. LOS	Exist. LOS	satfolw	width	lanes	volume
D	D	E	1699	14	1	272
C	D+	D	3206	25	2	452
~	~	~	~	~	~	100

	volume	120	660	64
	lanes	1	2	~
	width	14	25	~
	satflow	1699	3245	~
Exist.	LOS	D	D+	~
Optim.	LOS	D	D	~
Prop.	LOS	E	C	~

Existing Cycle length= 120 sec

Phase								
G	21.4	42.6	23.6	32.4				

Optimized Cycle length= 120 sec

Phase								
G	28.2	34.9	15.9	12.4	28.6			

Proposed Cycle length= 115 sec

Phase								
G	26.0	21.0	14.0	13.0	13.0	28.0		

N

Legend

G = Stage Length (sec)

Diagram I-I-12: Ironwood Exsiting/Optimized Data

I-II. CORSIM Output – MOE

Control Strategy for Signalized Intersections

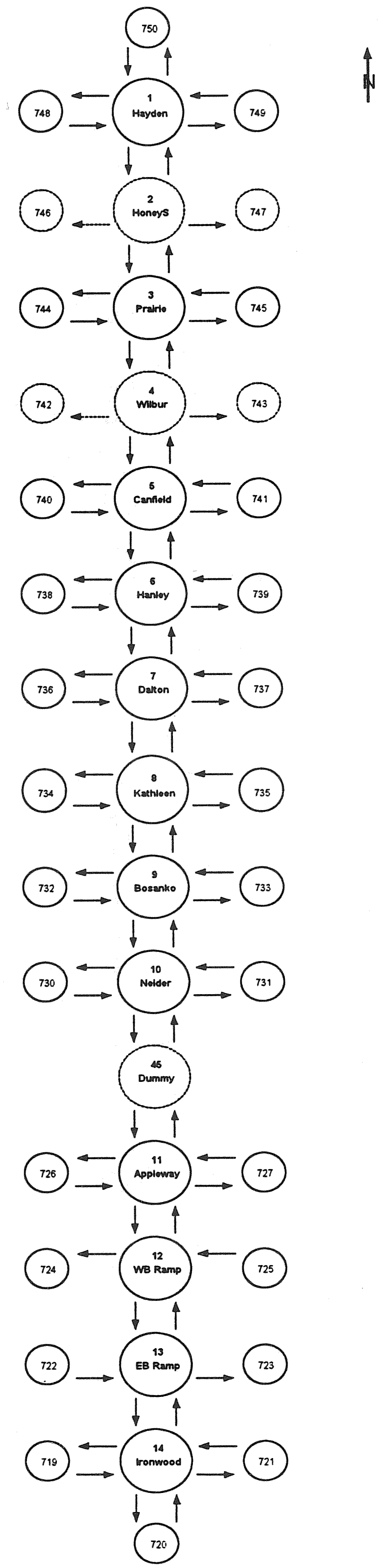


Table II-1: THRU SPEED (mph)

CORSIM Link		Plot Link	Northbound Existing	Northbound Optimized	Northbound Proposed
1	750	1	38.1	39.5	38.3
2	1	2	27.2	21.6	28.3
3	2	3	37.4	37	37.3
4	3	4	32.2	32	30.7
5	4	5	36.9	39.1	37
6	5	6	13.3	27.1	20.5
7	6	7	28.3	15.8	31.8
8	7	8	20.1	19.1	28.6
9	8	9	24.6	18.6	26.2
10	9	10	25.1	26.3	27.6
45	10	11	17	8.9	22.4
11	45	12	27.5	28.2	28.8
12	11	13	5.1	12.3	8.7
13	12	14	4.3	16.9	23.2
14	13	15	10.1	4.5	17
720	14	16	23	20.7	22.5

CORSIM Link		Plot Link	Southbound Existing	Southbound Optimized	Southbound Proposed
750	1	1	29.6	34.4	30.5
1	2	2	26.2	26.9	38
2	3	3	29.5	31.6	33.2
3	4	4	38.4	38.2	38.3
4	5	5	19.4	31.6	33.5
5	6	6	17.5	15.4	15.4
6	7	7	27.4	18.6	30.9
7	8	8	19.5	20.8	22.1
8	9	9	14.3	24.4	21.1
9	10	10	16.4	20.2	21.6
10	45	11	28.6	30	29.1
45	11	12	9.1	12.1	18.7
11	12	13	4.8	5.9	4
12	13	14	22.8	26.4	27.1
13	14	15	10.8	9.5	18.7
14	720	16	30.8	30.7	30.2

Table II-2: THRU DELAY TIME (sec/veh)

CORSIM Link		Plot Link	Northbound Existing	Northbound Optimized	Northbound Proposed
1	750	1	10.9	8.4	10.6
2	1	2	26.5	43.8	23.8
3	2	3	8.1	8.8	8.4
4	3	4	13.4	13.6	15.6
5	4	5	6.3	4.3	6.1
6	5	6	44	12.1	22.1
7	6	7	23.7	58.6	16.6
8	7	8	49.5	54.1	22.9
9	8	9	16.5	28.3	14.3
10	9	10	20.2	18.1	16
45	10	11	18.9	46.6	11.6
11	45	12	8.2	7.2	6.4
12	11	13	159.8	15.9	26.2
13	12	14	76.2	11.3	5.3
14	13	15	60.8	167.7	26
720	14	16	40.1	53.1	42.8

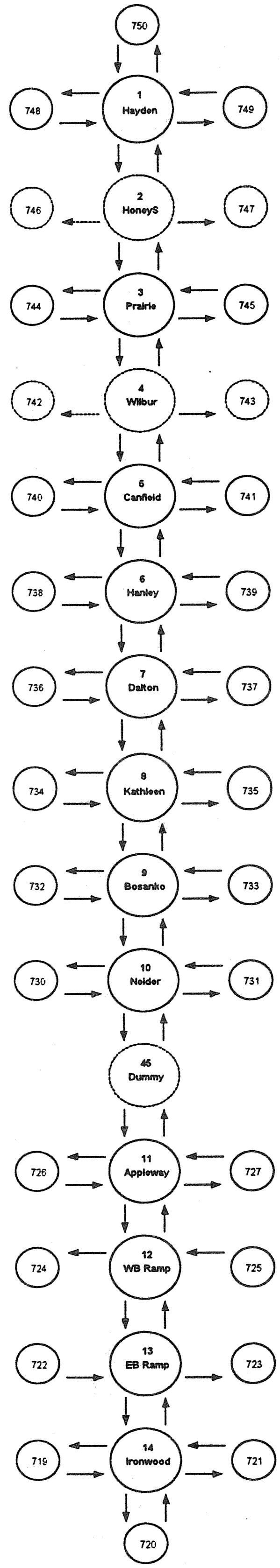
CORSIM Link		Plot Link	Southbound Existing	Southbound Optimized	Southbound Proposed
750	1	1	31.5	18.6	28.7
1	2	2	8.7	7.1	7.4
2	3	3	21.1	17	14.3
3	4	4	5.7	5.9	5.9
4	5	5	37.6	12.2	9.9
5	6	6	29.2	35.6	35.8
6	7	7	25.9	57	18.3
7	8	8	52.4	46.8	41.6
8	9	9	42.5	16.8	22.5
9	10	10	44.6	31.4	27.8
10	45	11	3.2	2.3	2.9
45	11	12	88.4	58.8	26.8
11	12	13	53.9	42.4	66.9
12	13	14	5.6	3.4	3
13	14	15	55.3	67.1	21.4
14	720	16	10.1	10.2	11.9

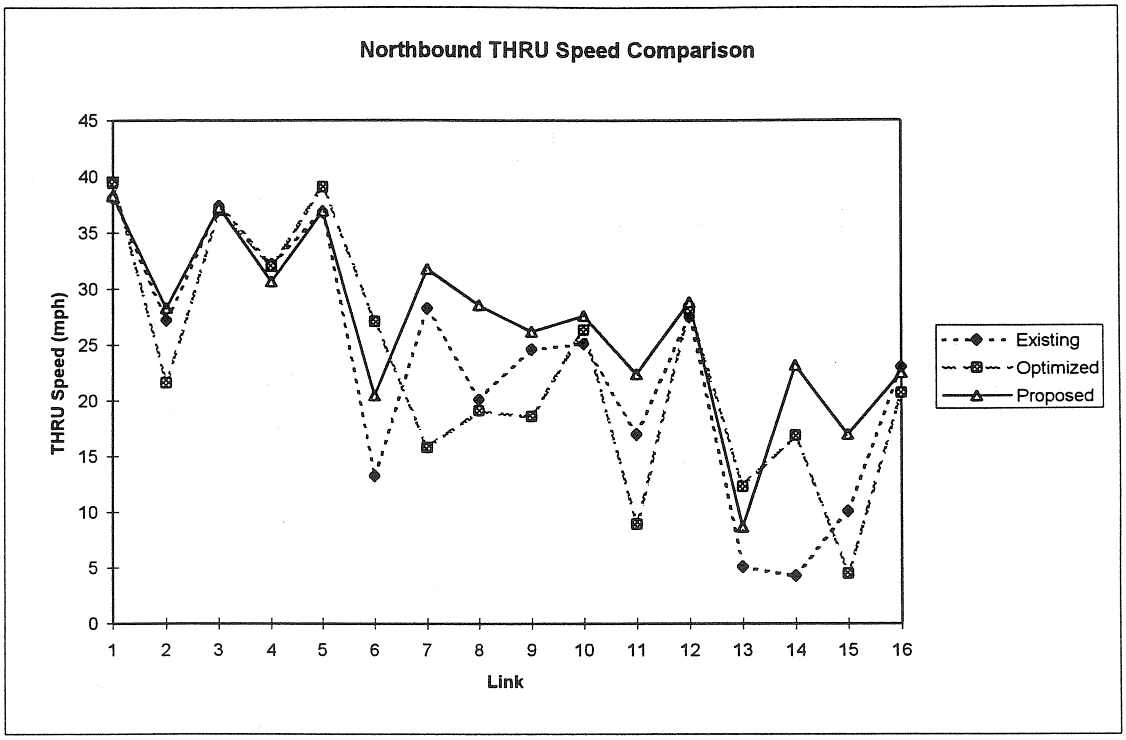
Table II-3: Queue Length (feet)

CORSIM Link		Plot Link	Northbound								
			Existing			Optimized			Proposed		
			TH	RT	LT	TH	RT	LT	TH	RT	LT
1	750	1	0	0	0	0	0	0	0	0	0
2	1	2	81	0	0	150	0	57	0	0	0
3	2	3	0	0	0	0	0	0	0	0	0
4	3	4	0	0	0	28	0	0	0	0	0
5	4	5	0	0	0	0	0	0	0	0	0
6	5	6	120	0	0	10	0	0	0	17	0
7	6	7	36	0	0	176	0	57	111	0	106
8	7	8	94	0	0	150	0	93	35	0	19
9	8	9	0	0	76	122	0	74	0	0	38
10	9	10	796	0	258	568	0	277	48	0	19
45	10	11	158	0	74	218	0	110	0	0	55
11	45	12	0	0	0	0	0	0	0	0	0
12	11	13	237	0	0	25	0	112	0	0	0
13	12	14	463	0	0	75	0	57	0	0	55
14	13	15	423	0	0	1188	0	0	0	0	0
720	14	16	148	0	57	205	0	0	28	0	0

CORSIM Link		Plot Link	Southbound								
			Existing			Optimized			Proposed		
			TH	RT	LT	TH	RT	LT	TH	RT	LT
750	1	1	0	0	38	521	19	95	0	0	36
1	2	2	0	0	0	0	0	0	0	0	0
2	3	3	0	0	0	74	0	0	0	0	0
3	4	4	0	0	0	0	0	0	0	0	0
4	5	5	57	0	27	10	0	110	10	0	76
5	6	6	162	0	86	55	0	110	101	0	57
6	7	7	61	0	0	213	57	38	0	0	0
7	8	8	63	0	0	140	0	38	236	0	112
8	9	9	0	0	0	45	0	0	0	0	38
9	10	10	85	0	74	242	0	93	111	0	76
10	45	11	0	0	0	0	0	0	0	0	0
45	11	12	705	45	0	218	0	76	174	0	131
11	12	13	118	0	89	139	0	0	291	0	0
12	13	14	0	0	0	18	0	17	0	0	36
13	14	15	131	0	203	74	38	129	0	0	0
14	720	16	0	0	0	0	0	0	0	0	0

I-III. MOE Comparison


750
N
748
1
Hayden
749
746
2
HoneyS
747
744
3
Prairie
745
742
4
Wilbur
743
740
5
Canfield
741
738
6
Hanley
739
736
7
Dalton
737
734
8
Kathleen
735
732
9
Bosanko
733
730
10
Nelder
731
45
Dummy
726
11
Appleway
727
724
12
WB Ramp
725
722
13
EB Ramp
723
719
14
Ironwood
721
720


Northbound THRU Speed Comparison
THRU Speed (mph)
45
40
35
30
25
20
15
10
5
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Existing
Optimized
Proposed

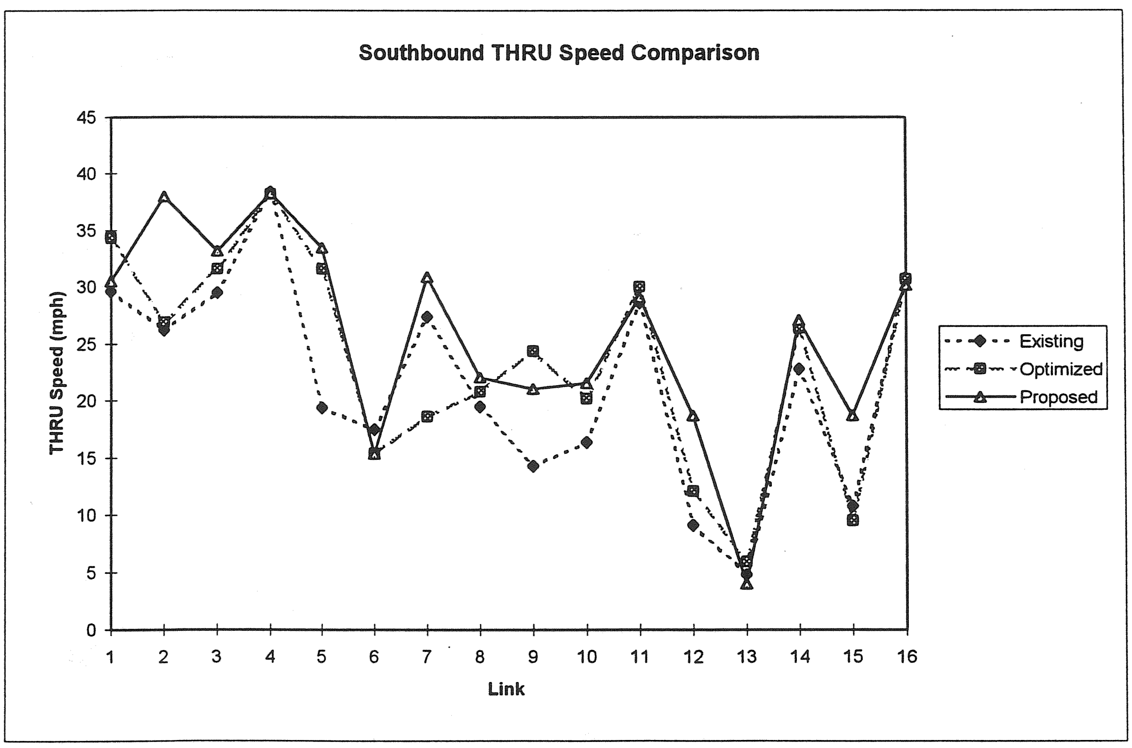
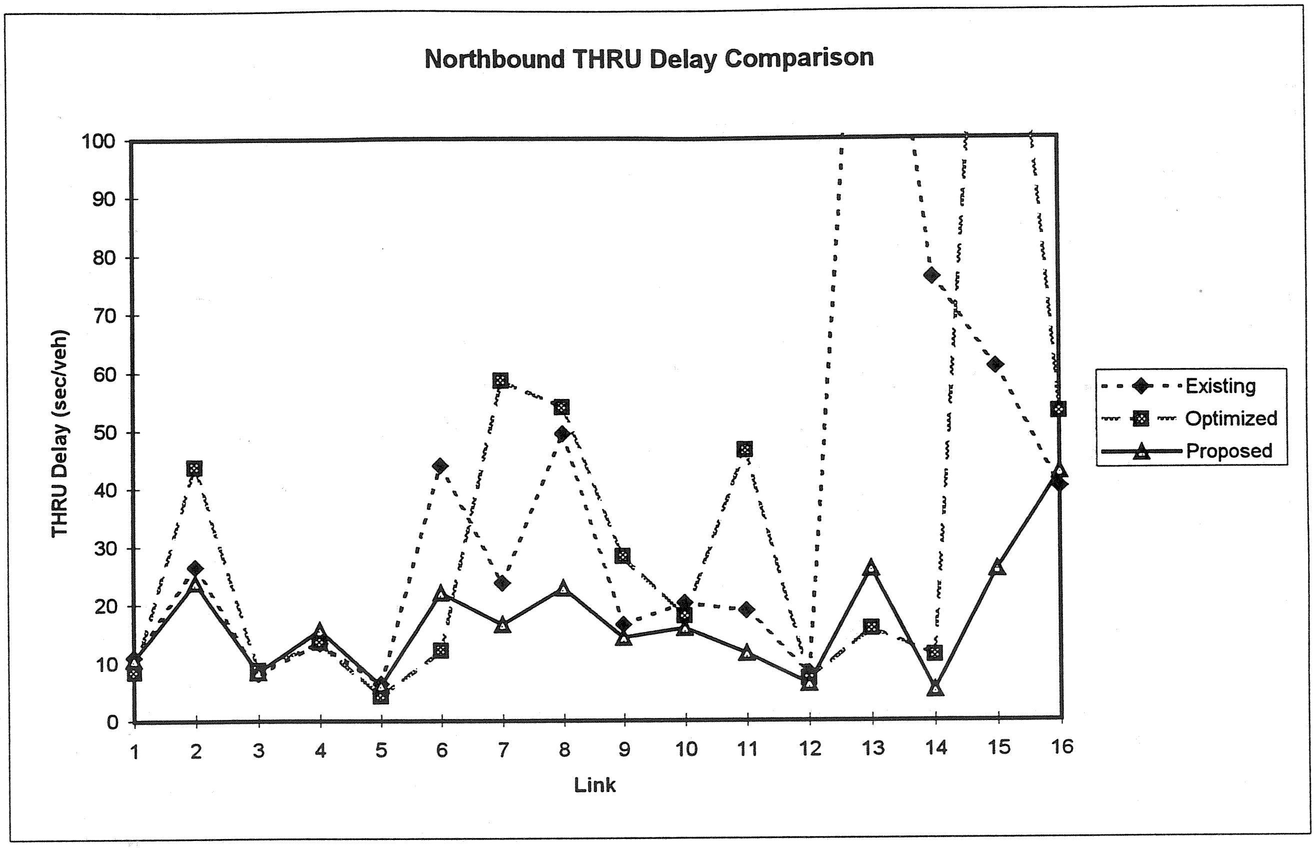

Southbound THRU Speed Comparison
THRU Speed (mph)
45
40
35
30
25
20
15
10
5
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Existing
Optimized
Proposed

Figure III-1


Northbound THRU Delay Comparison
THRU Delay (sec/veh)
0
10
20
30
40
50
60
70
80
90
100
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Existing
Optimized
Proposed

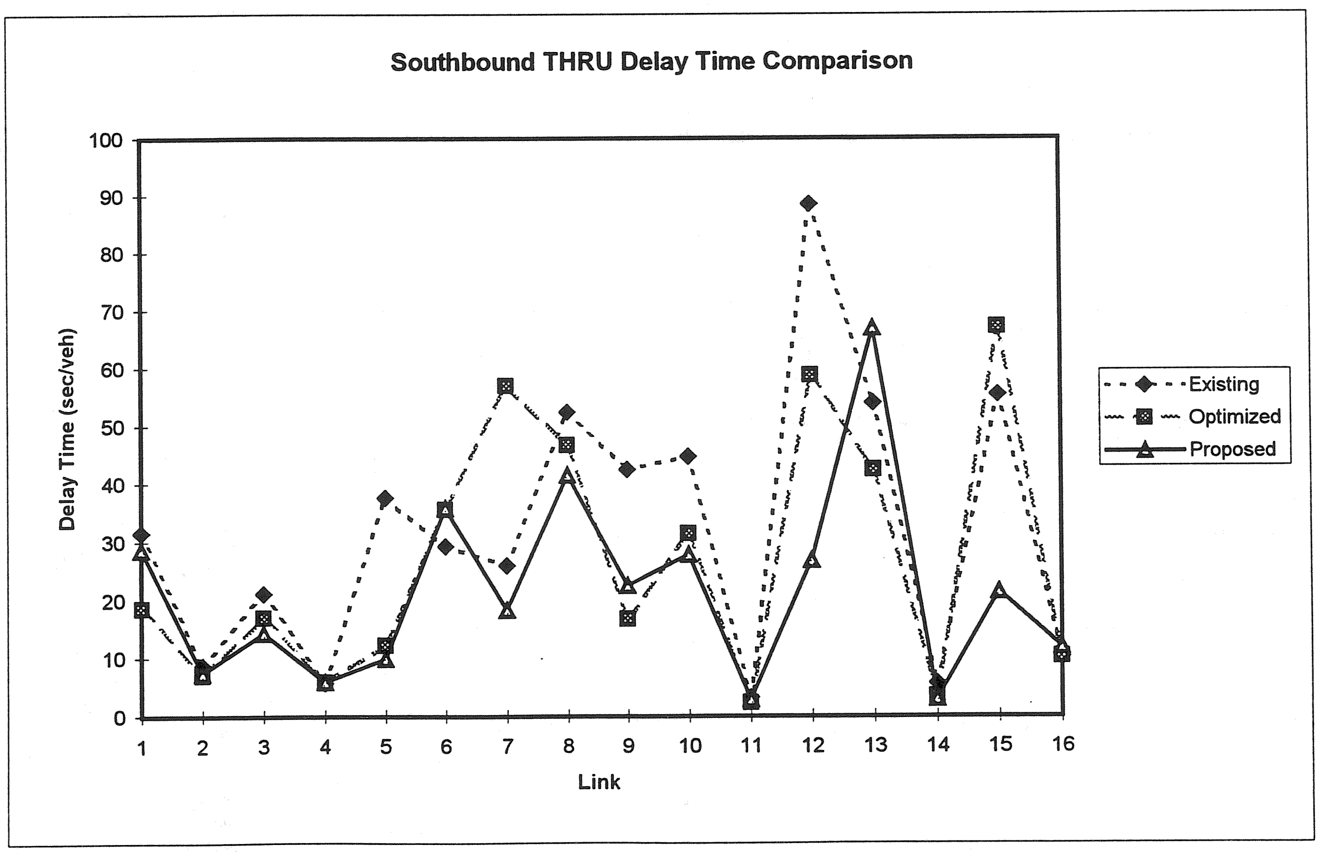
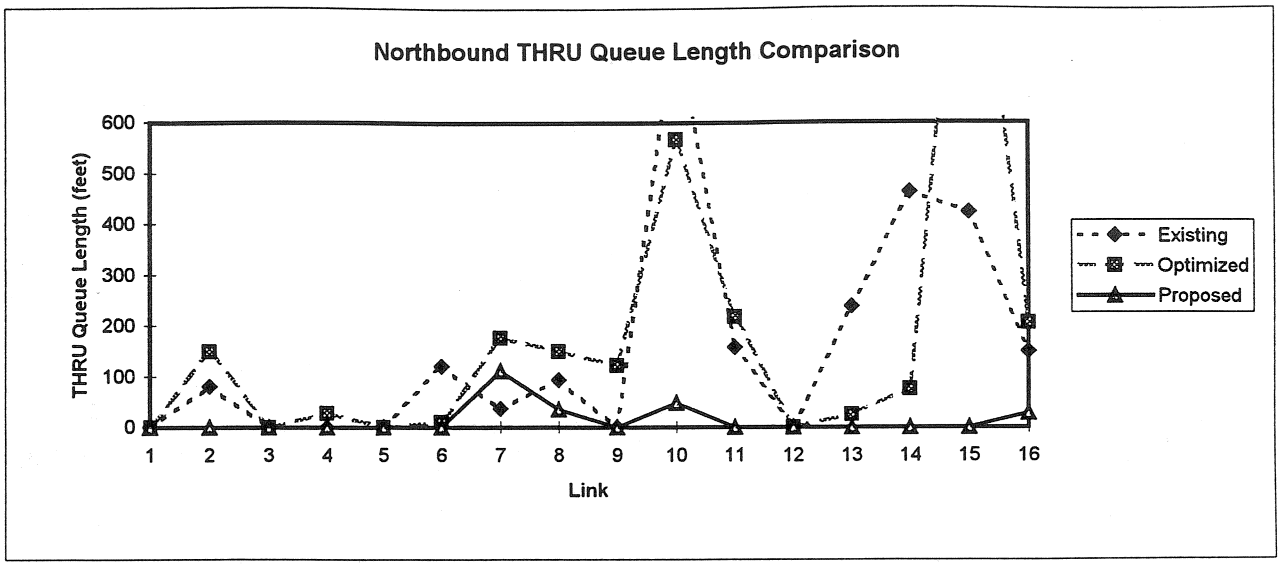
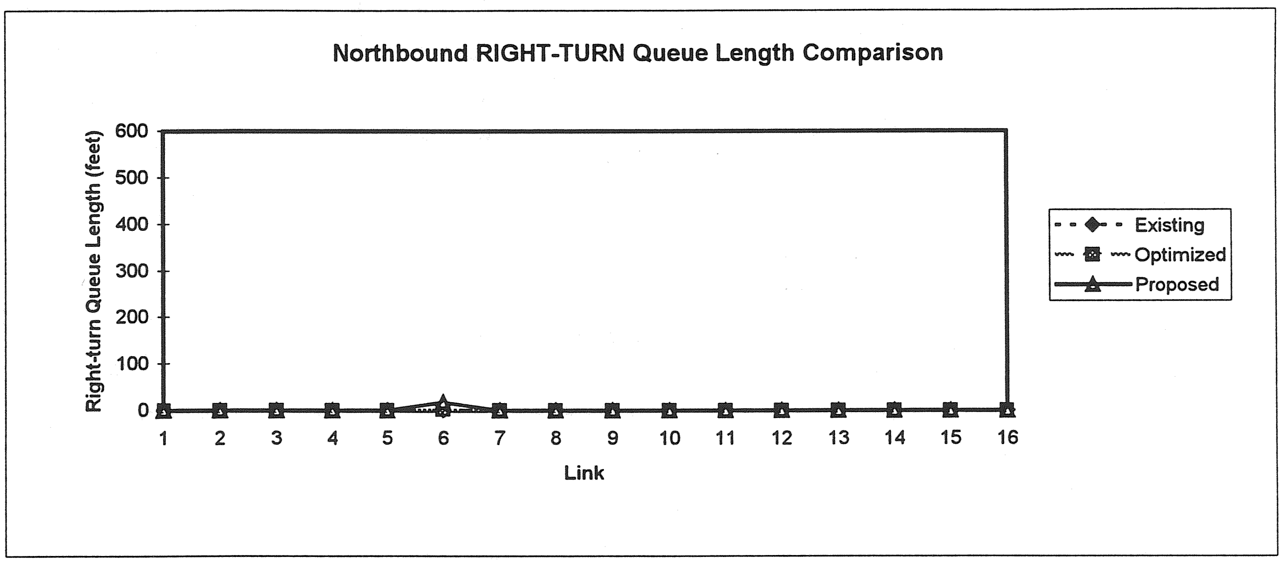

Southbound THRU Delay Time Comparison
Delay Time (sec/veh)
0
10
20
30
40
50
60
70
80
90
100
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Existing
Optimized
Proposed

Figure III-2


Northbound THRU Queue Length Comparison
THRU Queue Length (feet)
0
100
200
300
400
500
600
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Existing
Optimized
Proposed


Northbound RIGHT-TURN Queue Length Comparison
Right-turn Queue Length (feet)
0
100
200
300
400
500
600
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Existing
Optimized
Proposed

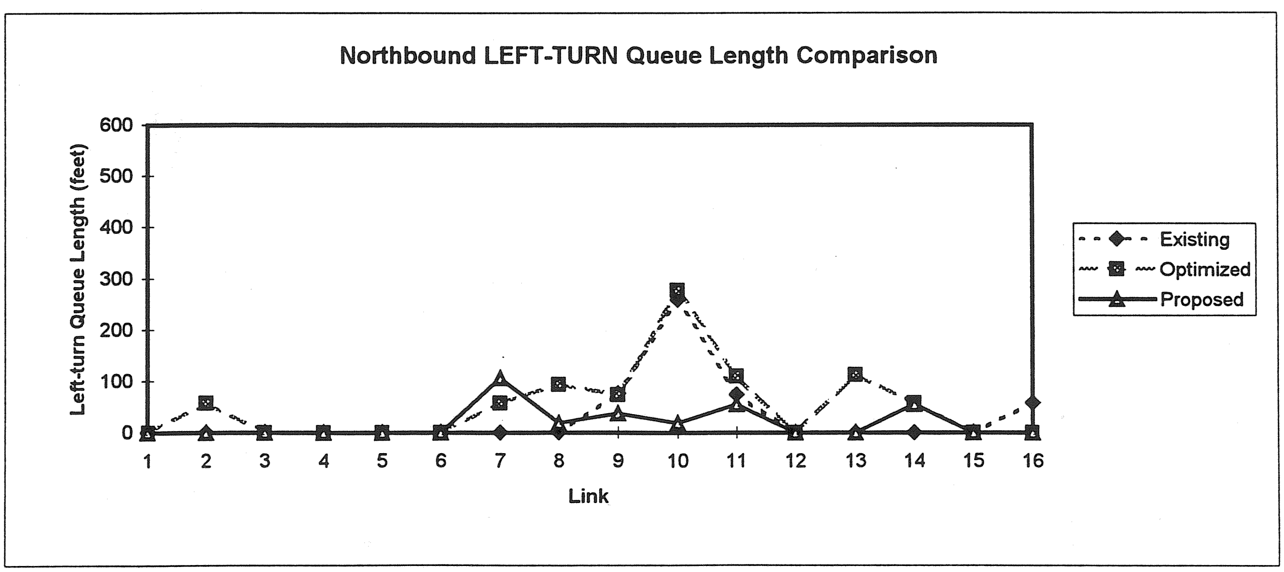
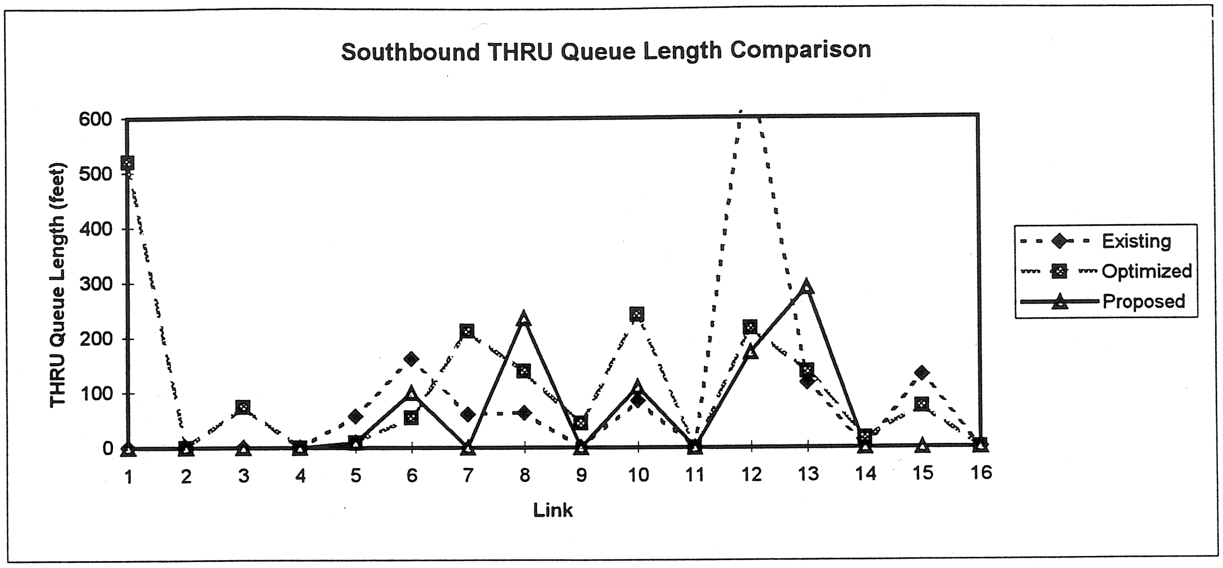
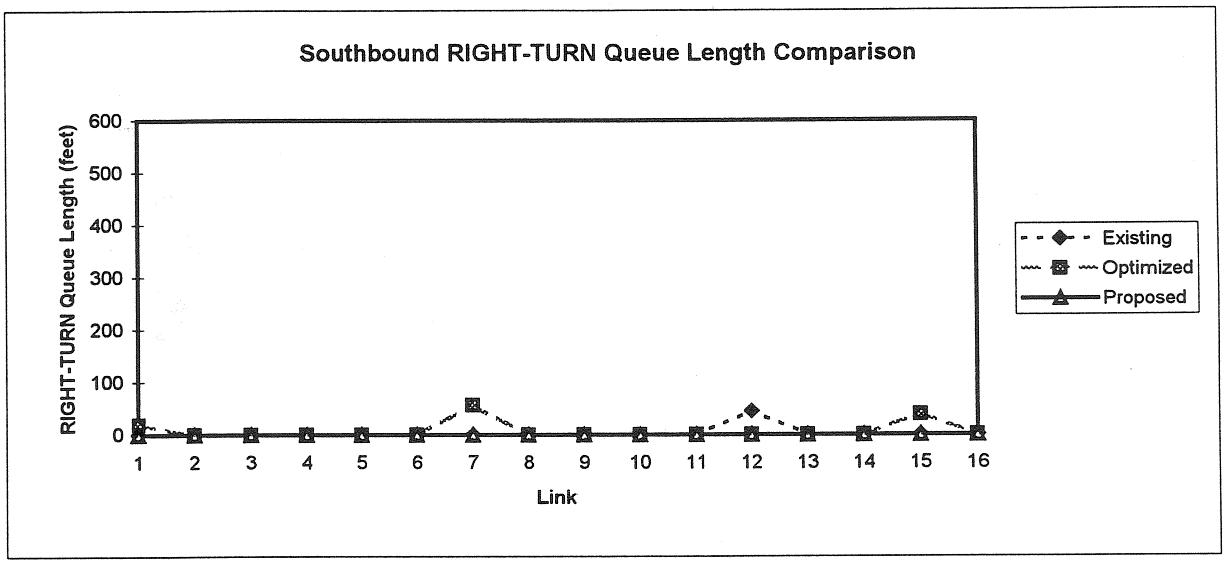

Northbound LEFT-TURN Queue Length Comparison
Left-turn Queue Length (feet)
0
100
200
300
400
500
600
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Existing
Optimized
Proposed

Figure III-3


Southbound THRU Queue Length Comparison
THRU Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Existing
Optimized
Proposed


Southbound RIGHT-TURN Queue Length Comparison
RIGHT-TURN Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Existing
Optimized
Proposed

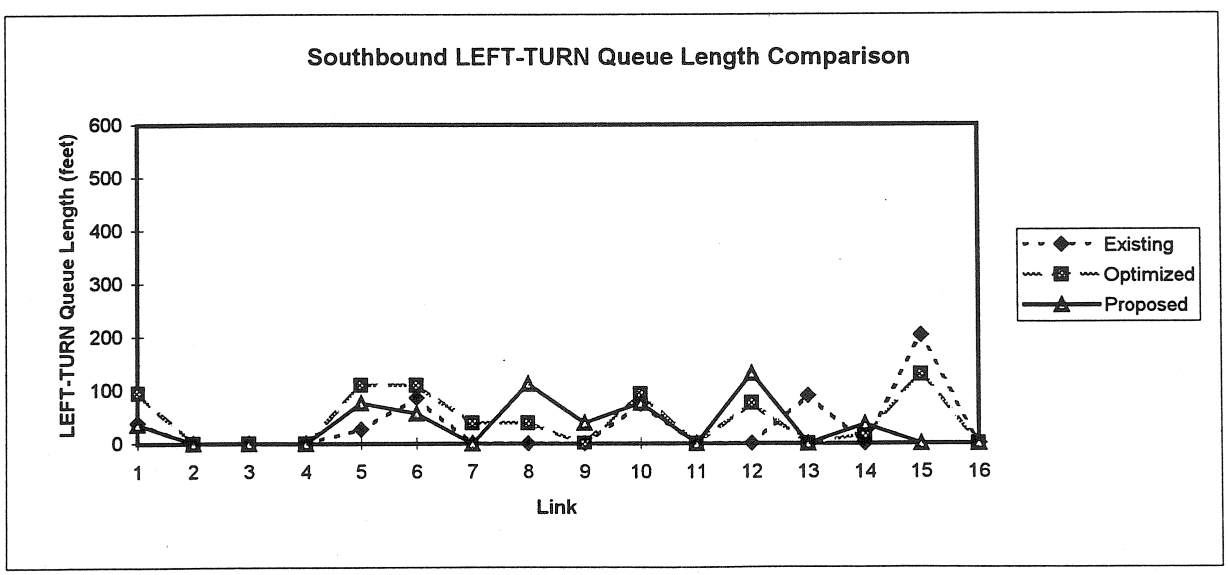
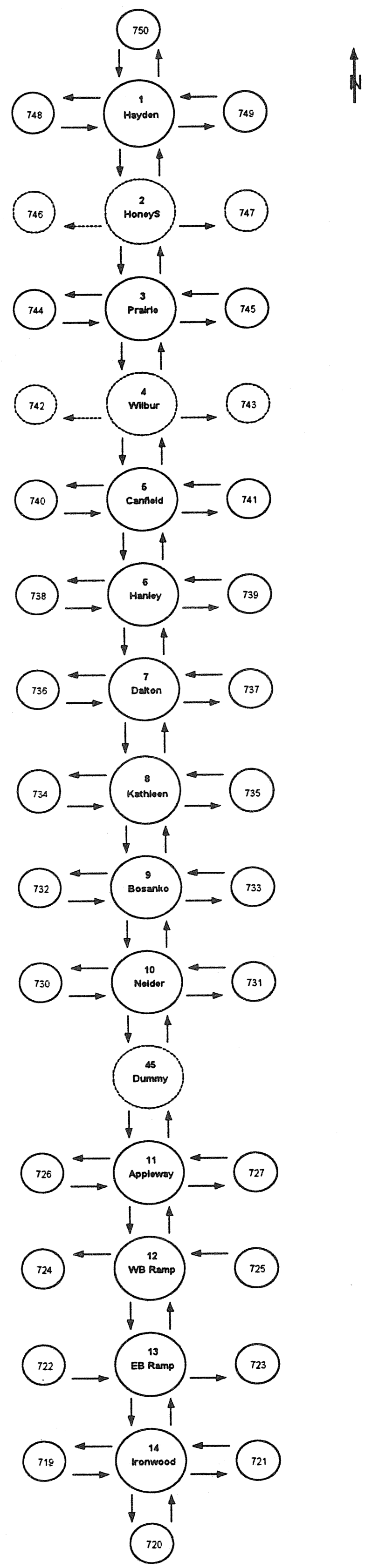
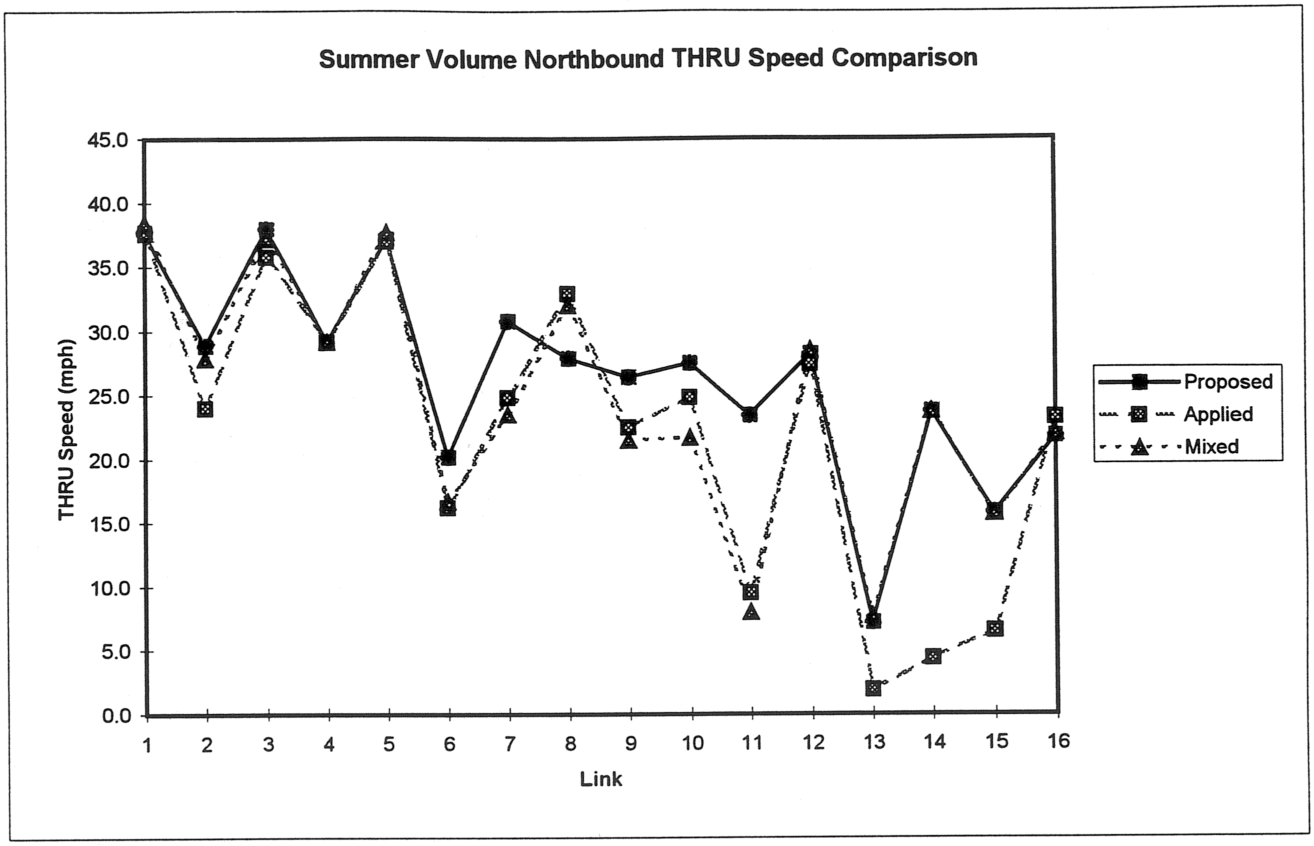
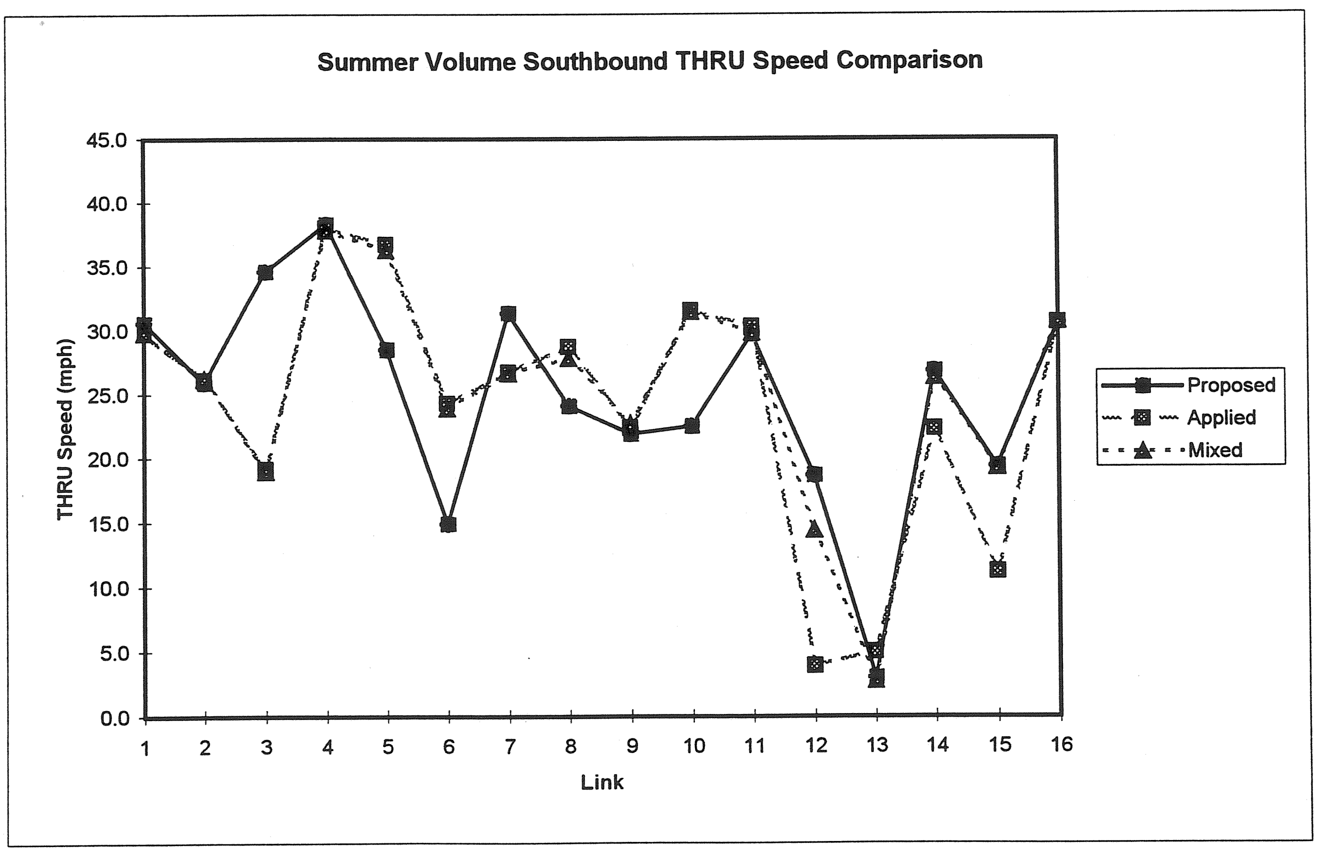

Southbound LEFT-TURN Queue Length Comparison
LEFT-TURN Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Existing
Optimized
Proposed

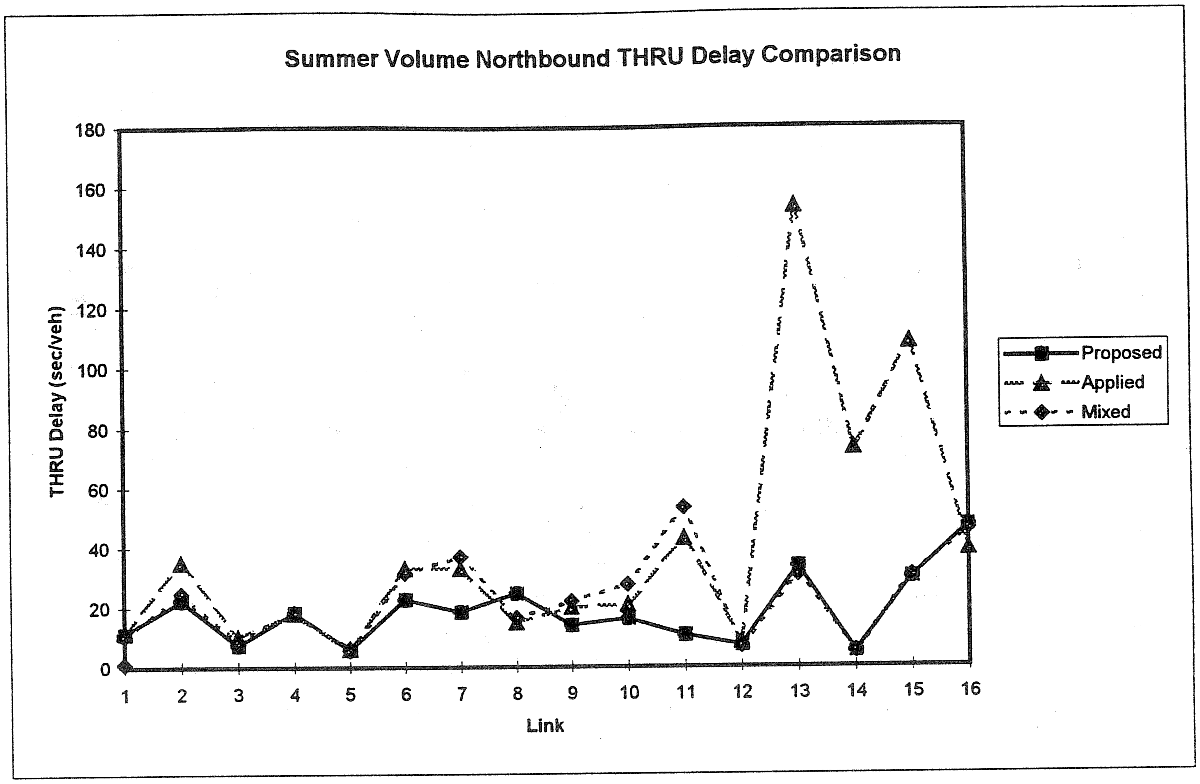
Figure III-4

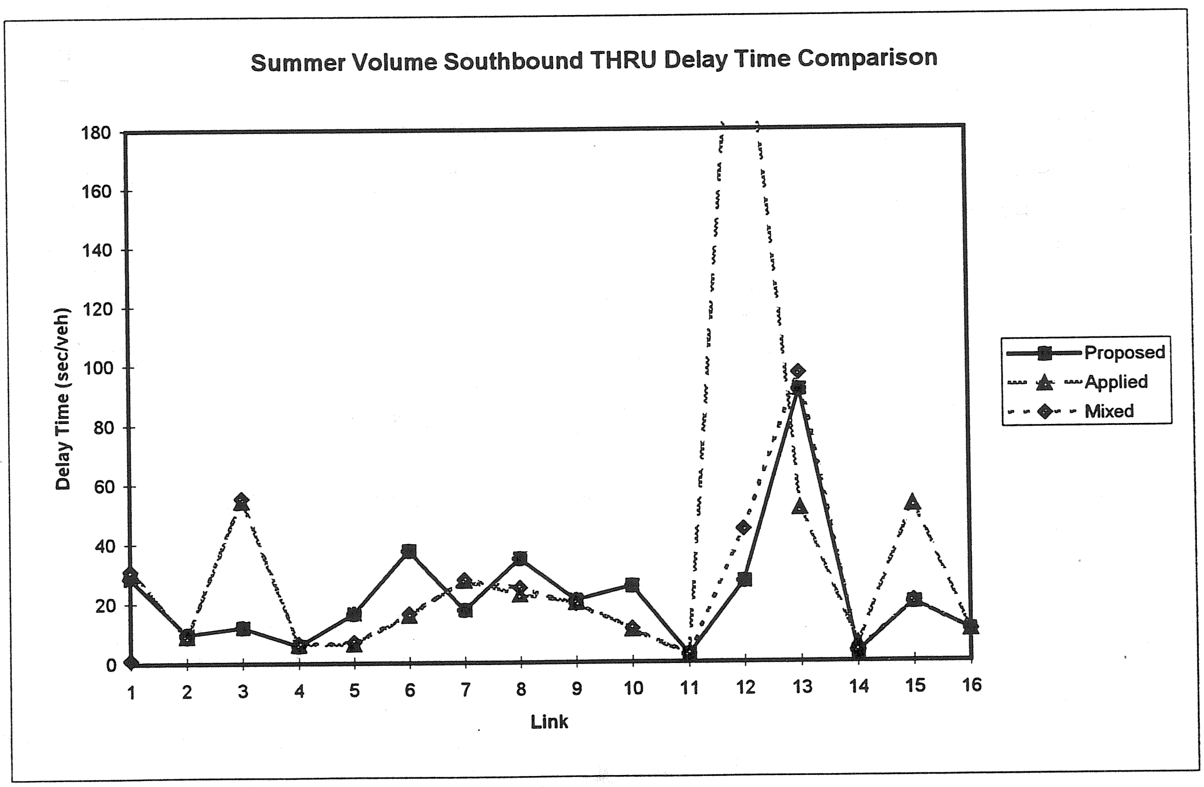
I-IV.MOE Percent Improvement

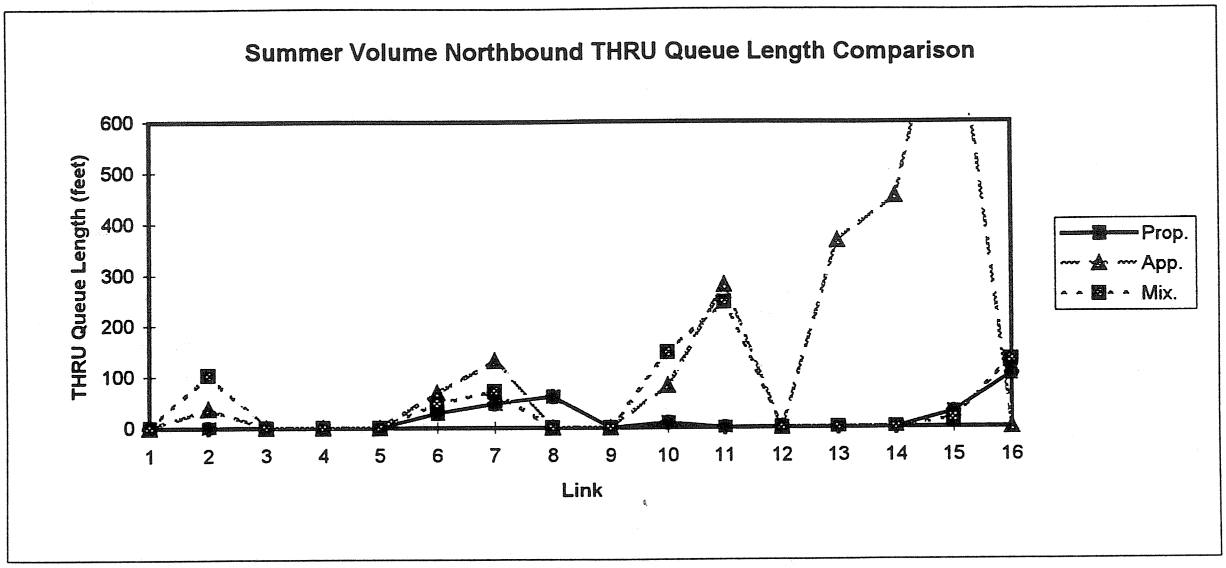

750
1
Hayden
748
749
2
HoneyS
746
747
3
Prairie
744
745
4
Wilbur
742
743
5
Canfield
740
741
6
Hanley
738
739
7
Dalton
736
737
8
Kathleen
734
735
9
Bosanko
732
733
10
Neider
730
731
45
Dummy
11
Appleway
726
727
12
WB Ramp
724
725
13
EB Ramp
722
723
14
Ironwood
719
721
720
N


Summer Volume Northbound THRU Speed Comparison
THRU Speed (mph)
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16
Link
Proposed
Applied
Mixed

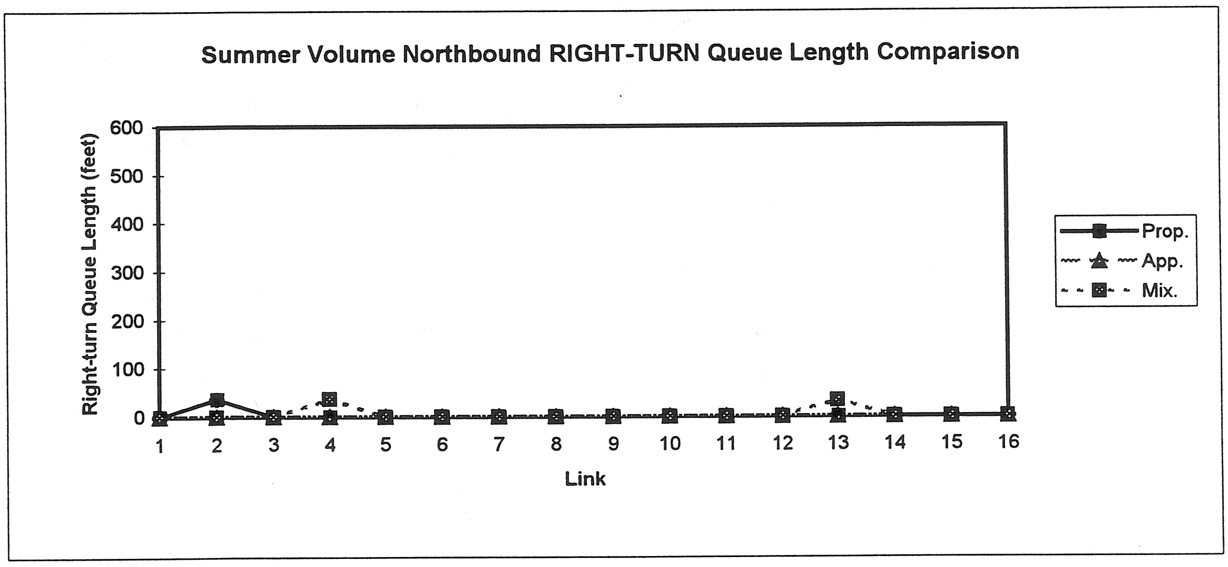

Summer Volume Southbound THRU Speed Comparison
THRU Speed (mph)
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16
Link
Proposed
Applied
Mixed


Summer Volume Northbound THRU Delay Comparison
THRU Delay (sec/veh)
0
20
40
60
80
100
120
140
160
180
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed

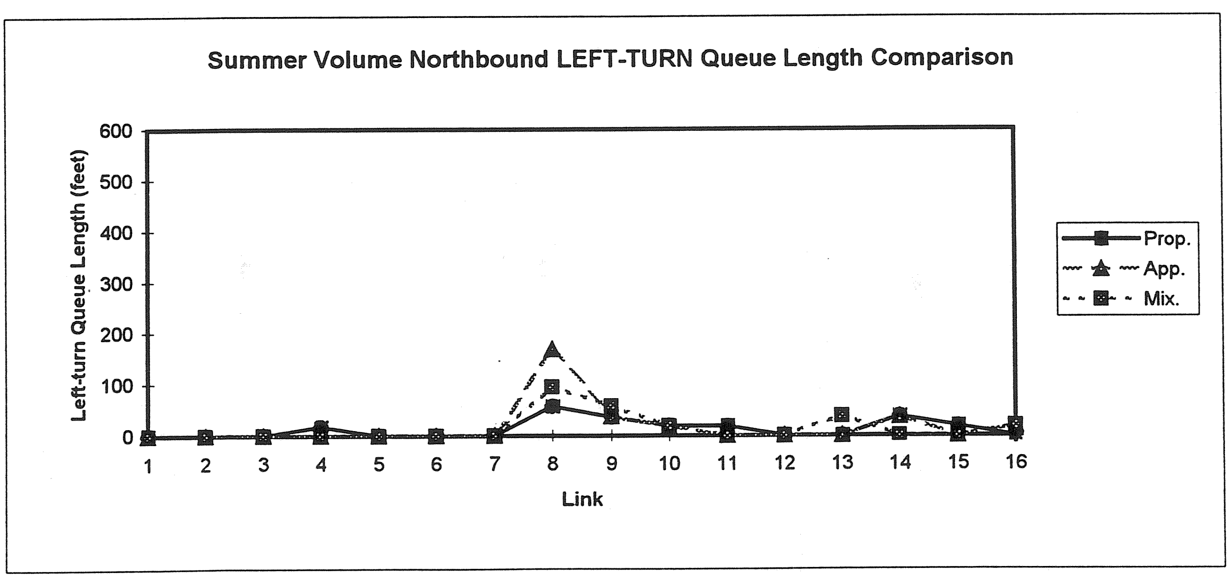

Summer Volume Southbound THRU Delay Time Comparison
Delay Time (sec/veh)
0
20
40
60
80
100
120
140
160
180
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed



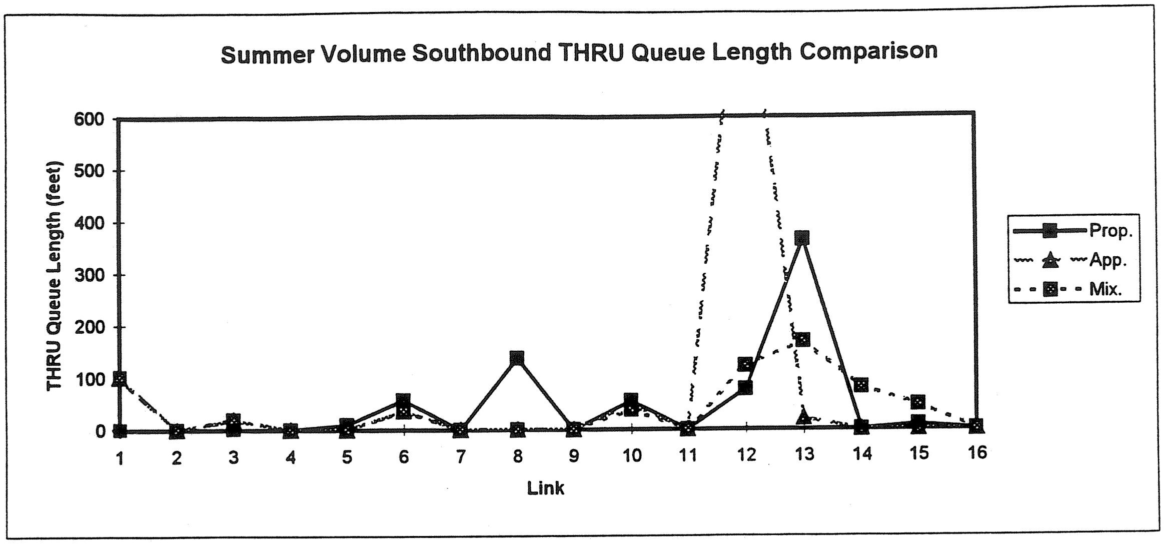
Summer Volume Northbound THRU Queue Length Comparison
THRU Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.

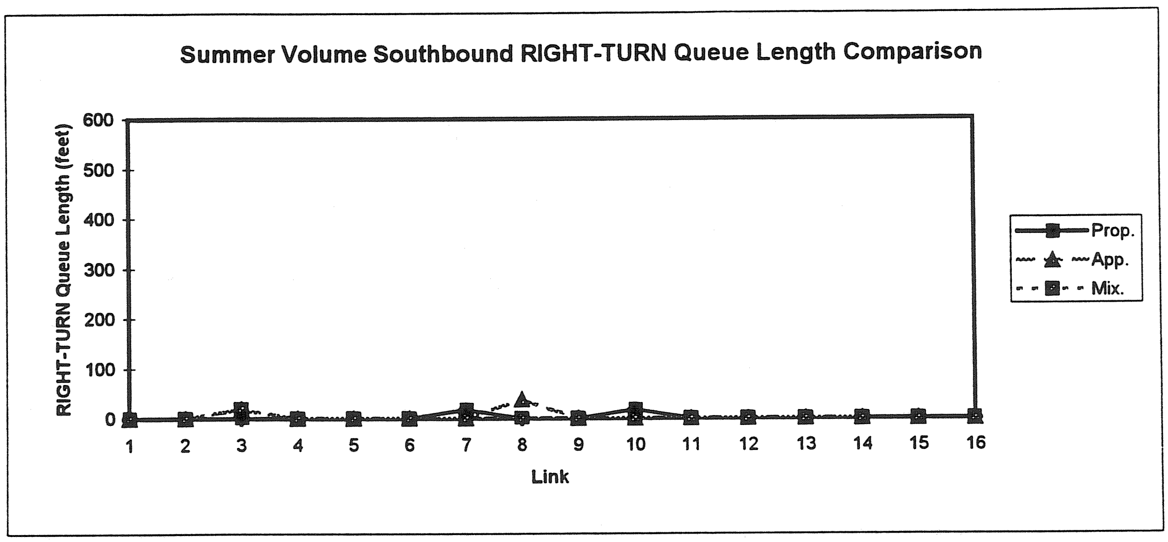


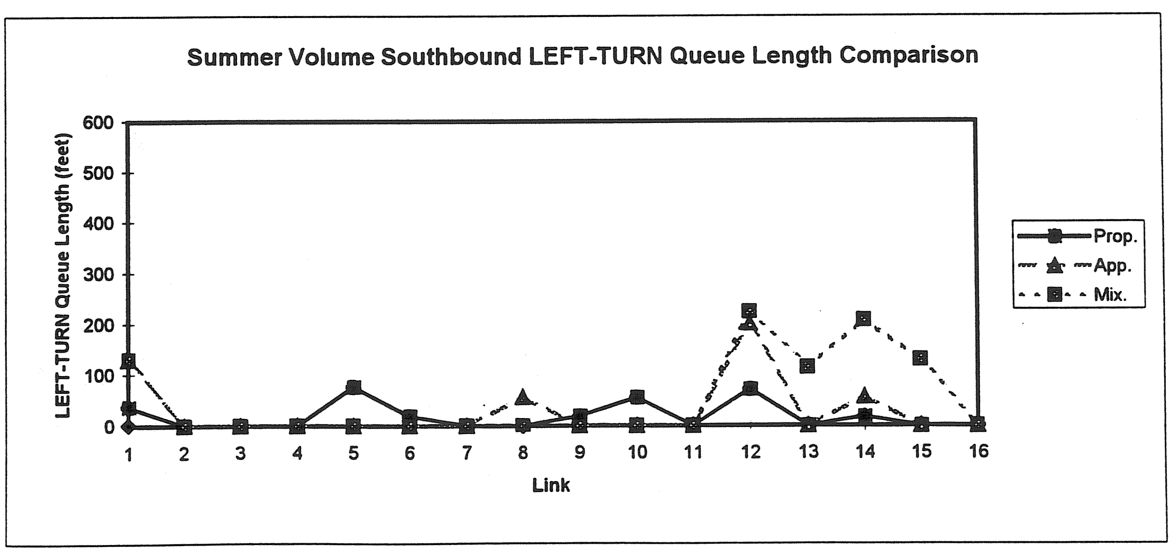
Summer Volume Northbound RIGHT-TURN Queue Length Comparison
Right-turn Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.



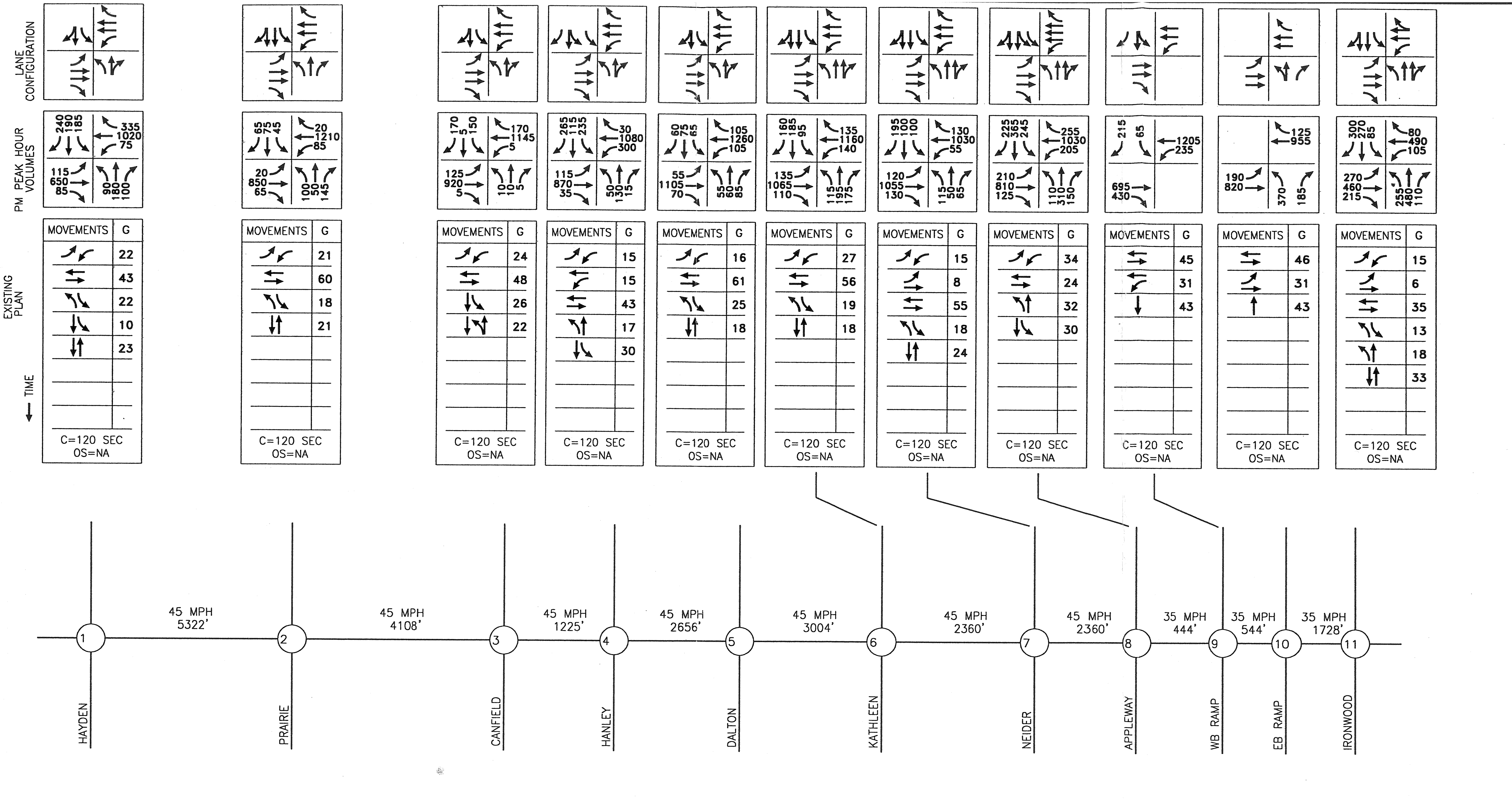
Summer Volume Northbound LEFT-TURN Queue Length Comparison
Left-turn Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.


Summer Volume Southbound THRU Queue Length Comparison
THRU Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.


Summer Volume Southbound RIGHT-TURN Queue Length Comparison
RIGHT-TURN Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.


Summer Volume Southbound LEFT-TURN Queue Length Comparison
LEFT-TURN Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.

I-V. Proposed Signal Timings for AM, Mid-day, and PM



LEGEND

C	=	CYCLE LENGTH (SEC)
G	=	STAGE LENGTH (SEC)
OS	=	OFFSET (SEC)
XX MPH	=	POSTED SPEED

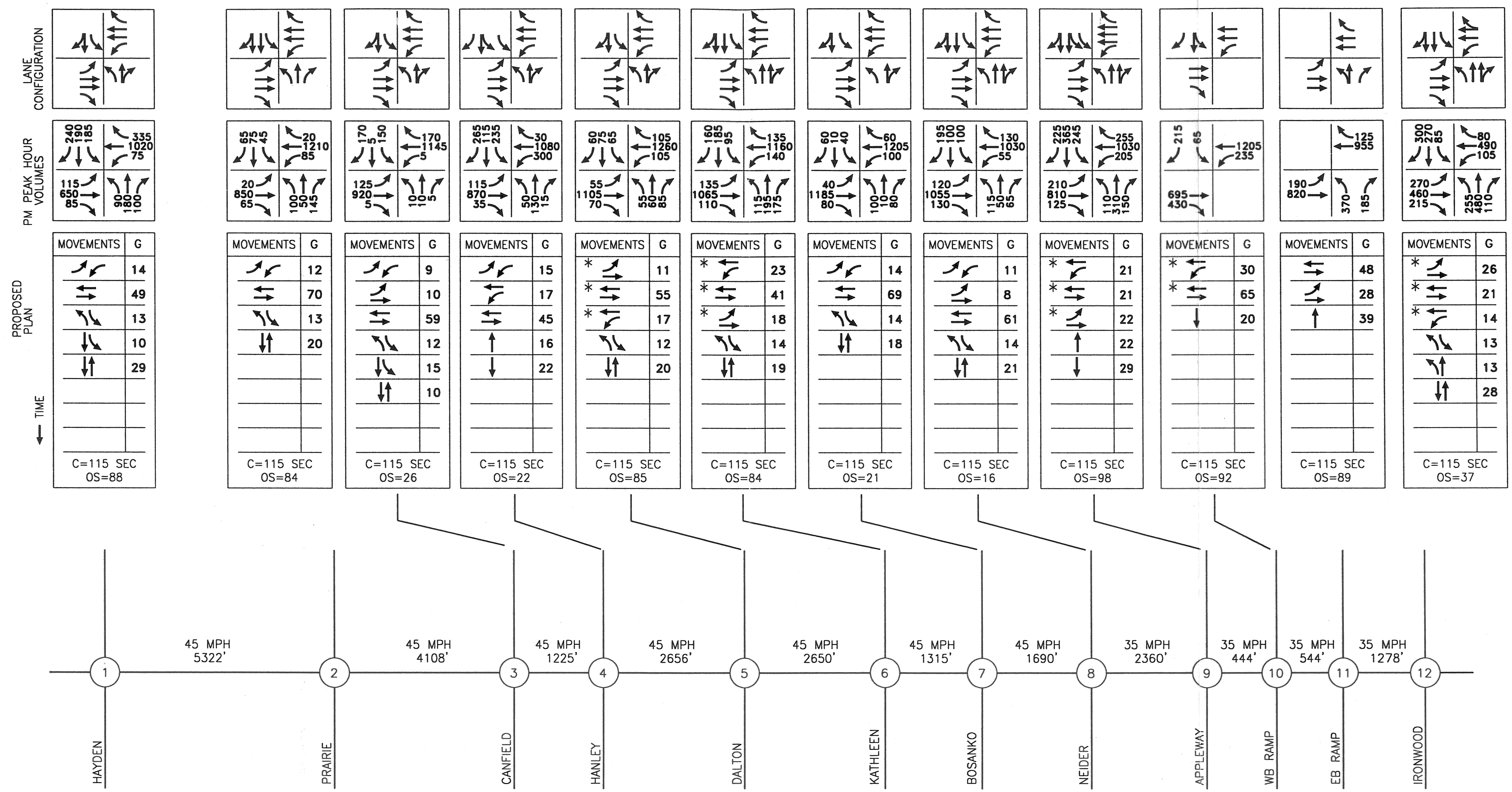
US HIGHWAY 95 SIGNAL RETIMING
COEUR d'ALENE, IDAHO

FIGURE
1

SIGNAL TIMING MAP
US HIGHWAY 95
EXISTING PM PEAK

MAY 1997

2268HX01



LEGEND

C = CYCLE LENGTH (SEC)
G = STAGE LENGTH (SEC)
OS = OFFSET (SEC)
XX MPH = POSTED SPEED

* CHANGE FROM EXISTING PHASING

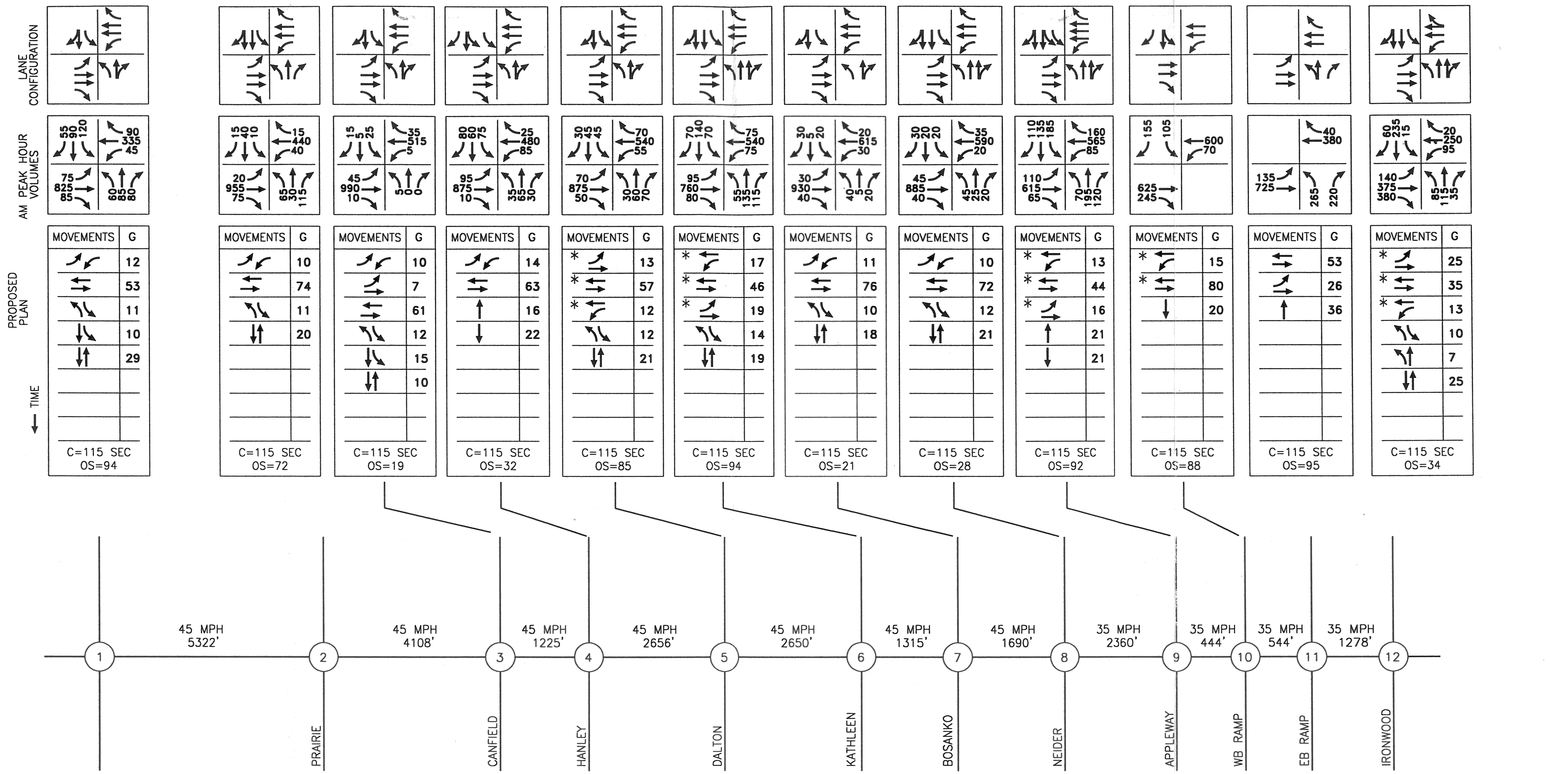
US HIGHWAY 95 SIGNAL RETIMING
COEUR d'ALENE, IDAHO

FIGURE 2

SIGNAL TIMING MAP
US HIGHWAY 95
OPTIMIZED PM PEAK

MAY 1997

226BHX02



LEGEND

C	=	CYCLE LENGTH (SEC)
G	=	STAGE LENGTH (SEC)
OS	=	OFFSET (SEC)
XX MPH	=	POSTED SPEED

* CHANGE FROM EXISTING PHASING

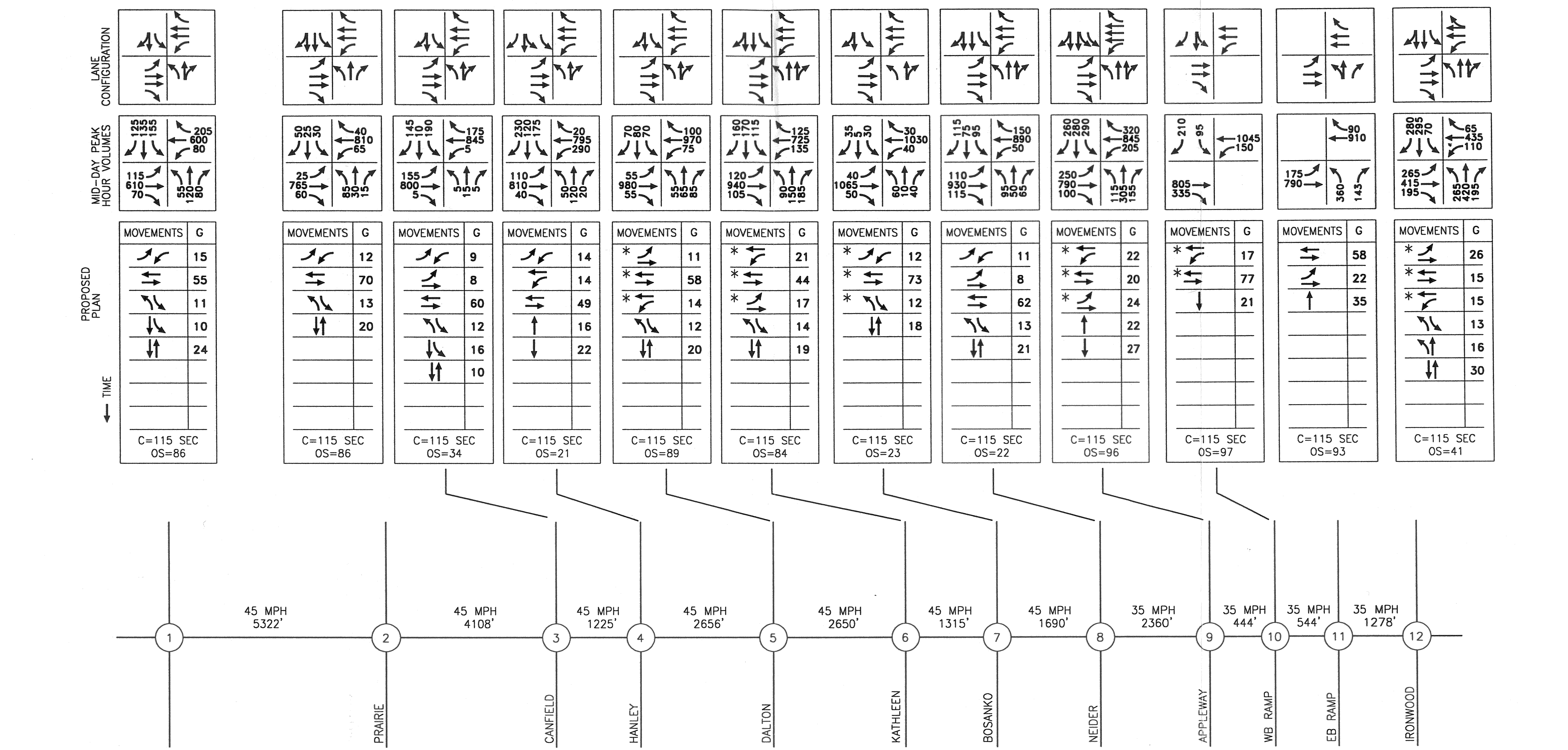
US HIGHWAY 95 SIGNAL RETIMING
COEUR d'ALENE, IDAHO

FIGURE 3

SIGNAL TIMING MAP
US HIGHWAY 95
OPTIMIZED AM PEAK

MAY 1997

2268HX03



LEGEND

C	=	CYCLE LENGTH (SEC)
G	=	STAGE LENGTH (SEC)
OS	=	OFFSET (SEC)
XX MPH	=	POSTED SPEED

* CHANGE FROM EXISTING PHASING

US HIGHWAY 95 SIGNAL RETIMING
COEUR d'ALENE, IDAHO

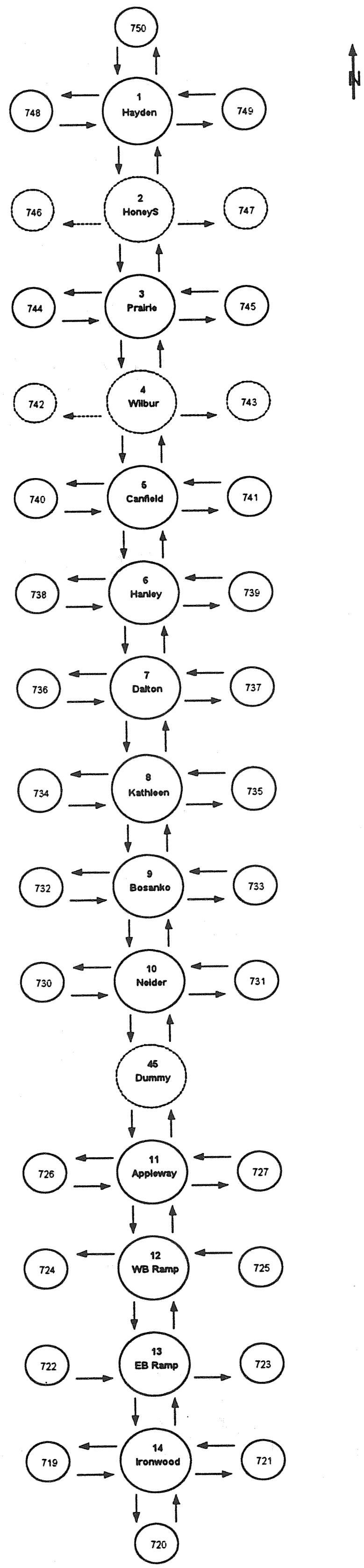
FIGURE 4

SIGNAL TIMING MAP
US HIGHWAY 95
OPTIMIZED MID-DAY PEAK

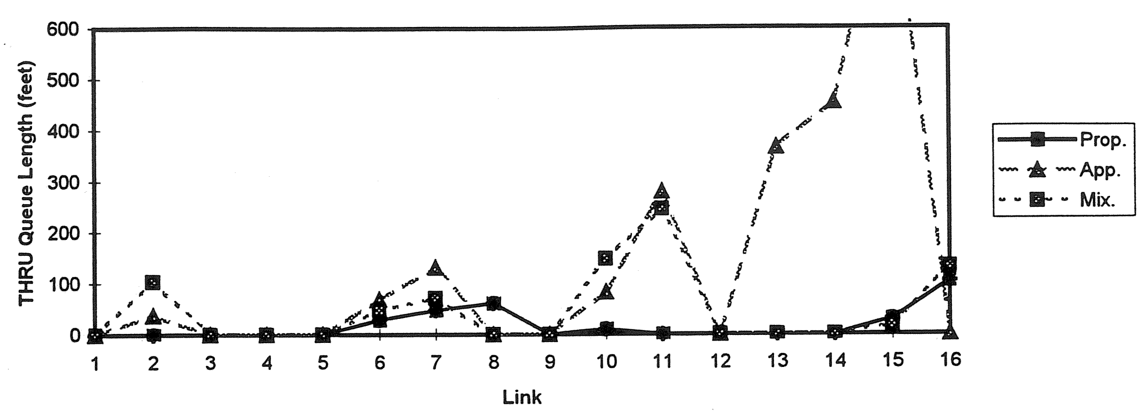
MAY 1997

2268HX04

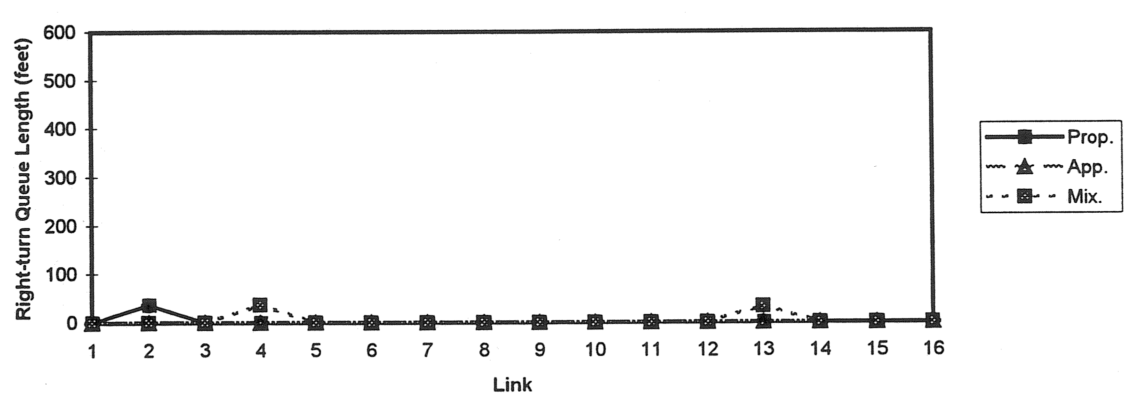
I-VI. PROPOSED, APPLIED, and MIXED Signal Timings Comparison under Summer Volume


750
1 Hayden
748
749
2 HoneyS
746
747
3 Prairie
744
745
4 Wilbur
742
743
5 Canfield
740
741
6 Hanley
738
739
7 Dalton
736
737
8 Kathleen
734
735
9 Bosanko
732
733
10 Neider
730
731
45 Dummy
11 Appleway
726
727
12 WB Ramp
724
725
13 EB Ramp
722
723
14 Ironwood
719
721
720
N

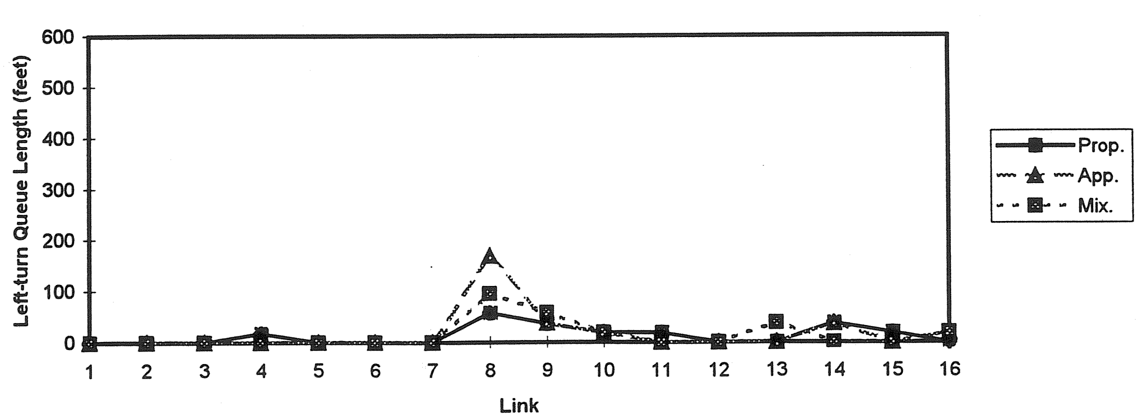
Summer Volume Northbound THRU Queue Length Comparison

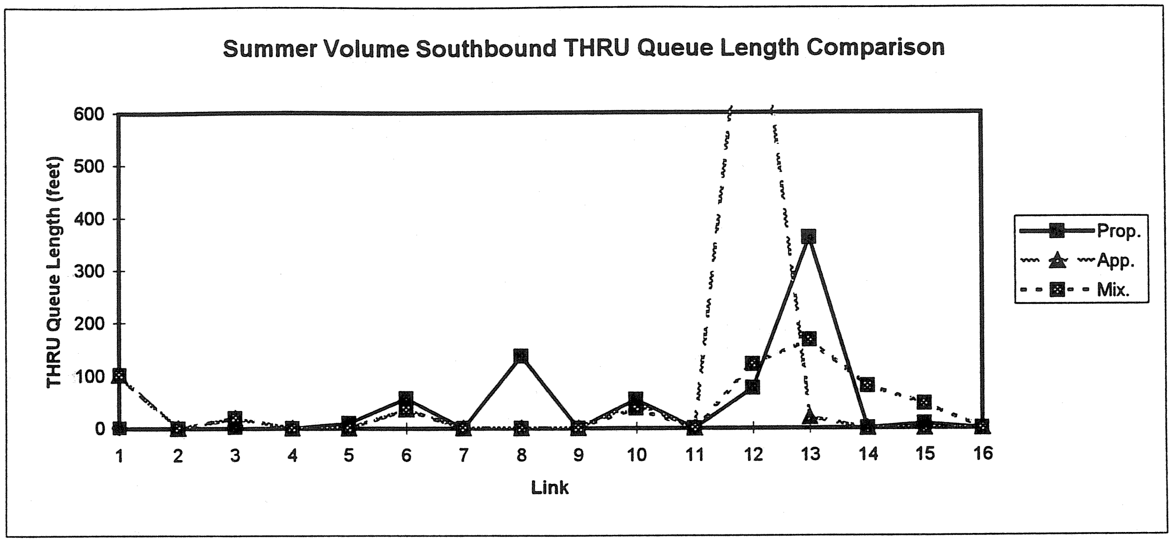


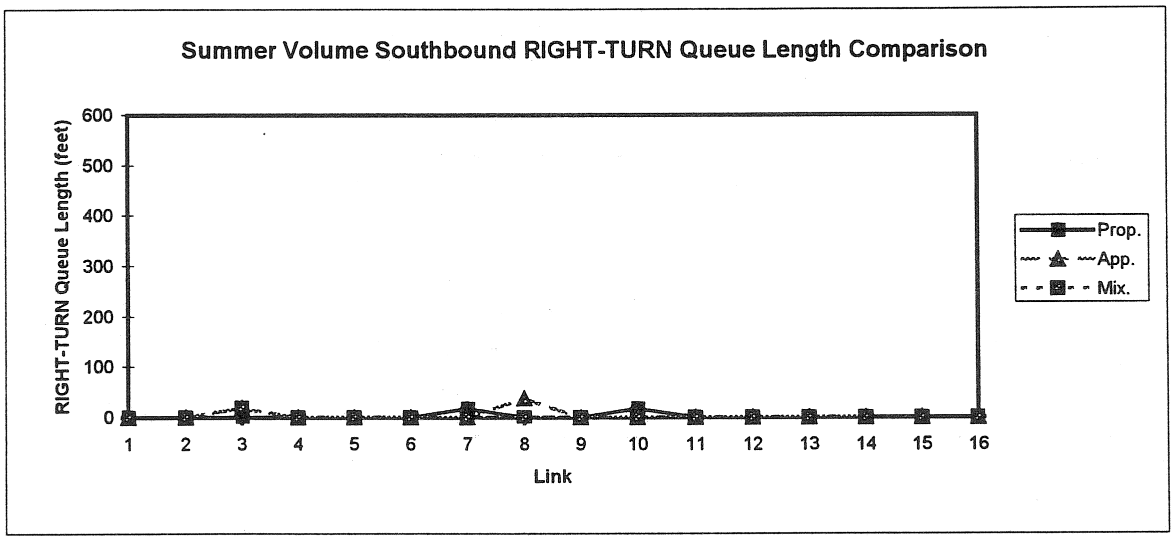
Summer Volume Northbound RIGHT-TURN Queue Length Comparison

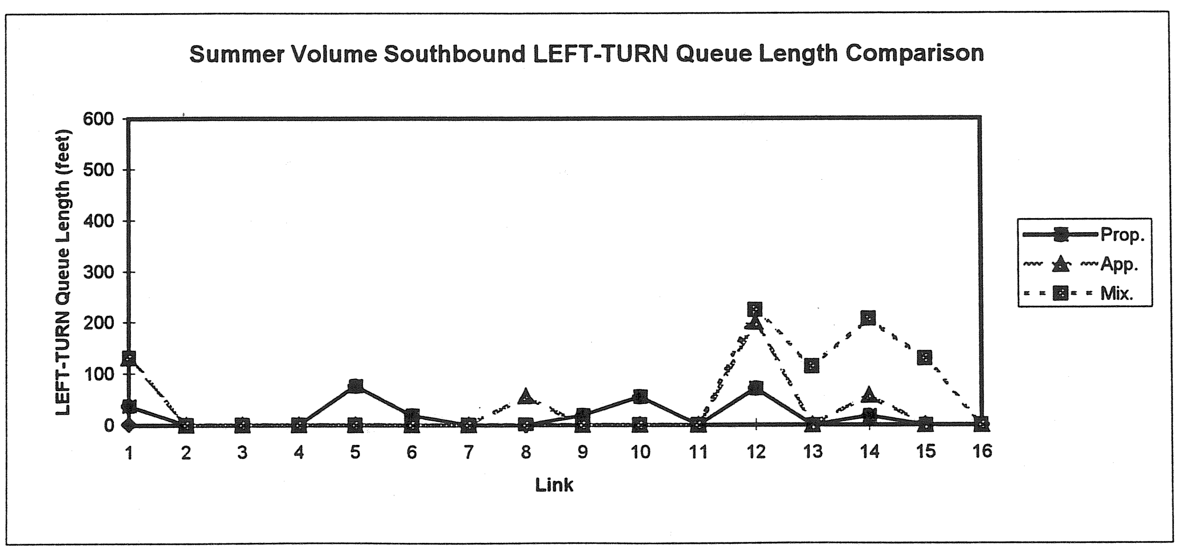


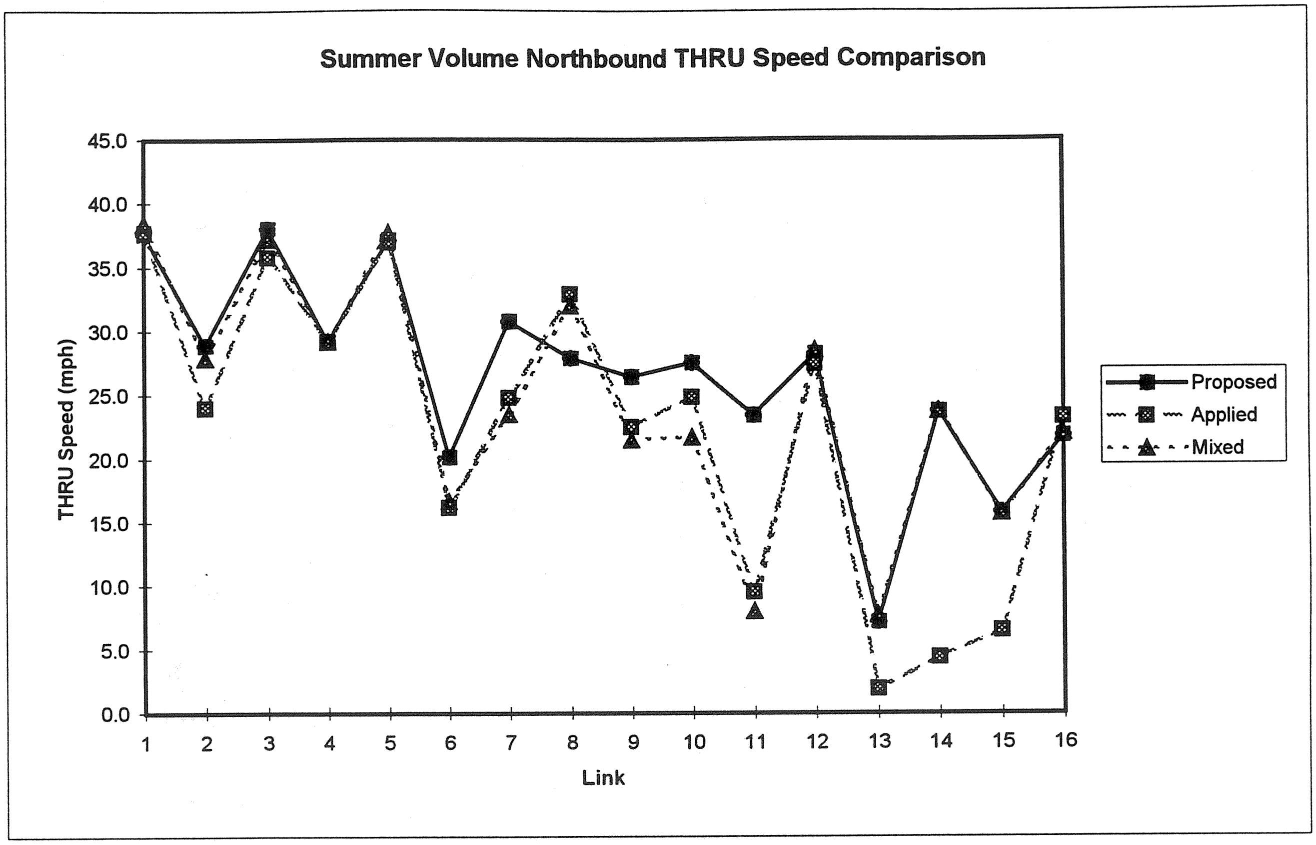
Summer Volume Northbound LEFT-TURN Queue Length Comparison

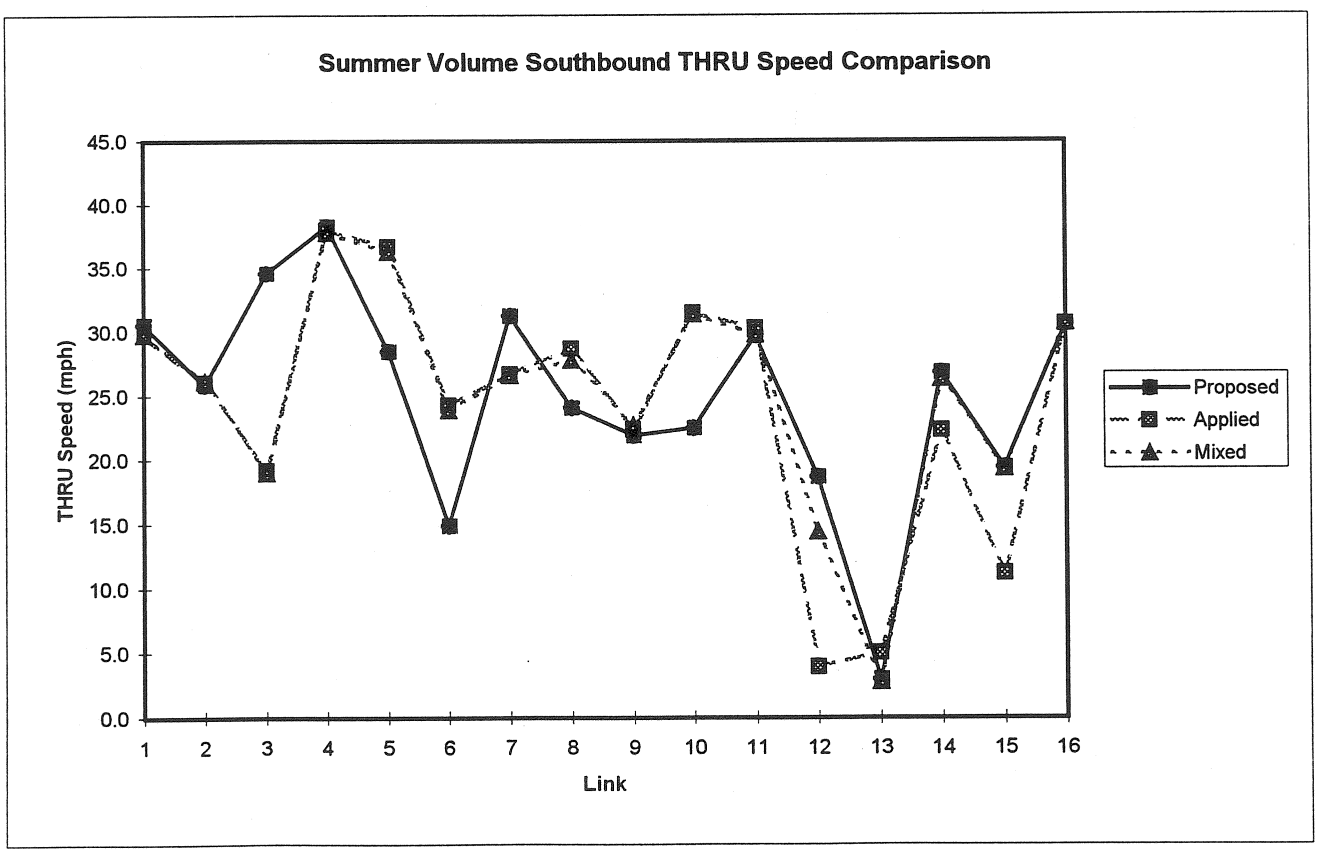


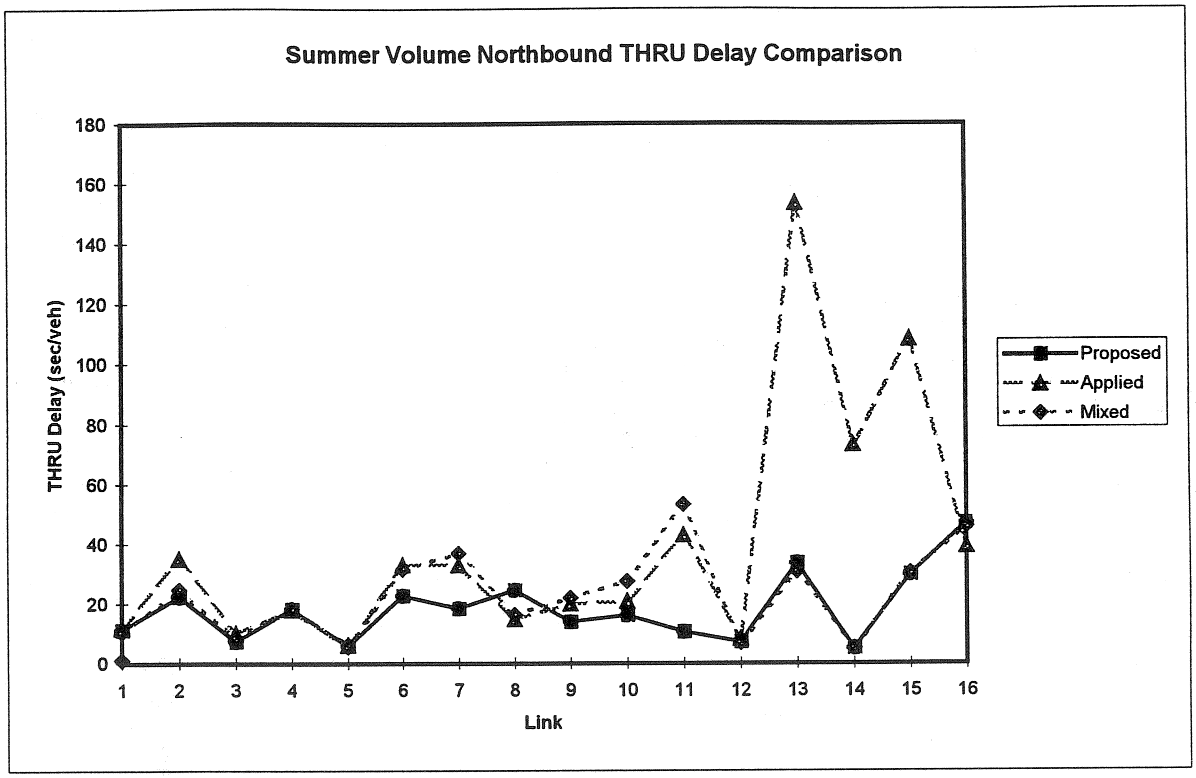

Summer Volume Southbound THRU Queue Length Comparison
THRU Queue Length (feet)
0
100
200
300
400
500
600
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.

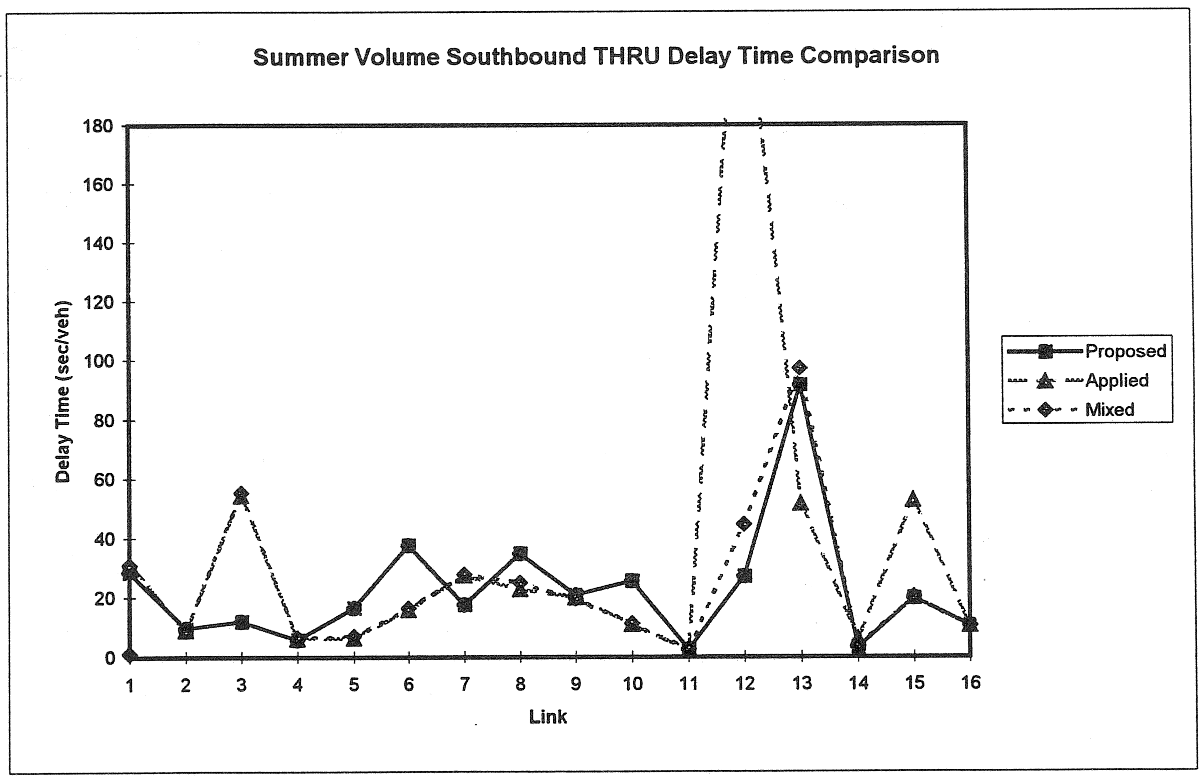

Summer Volume Southbound RIGHT-TURN Queue Length Comparison
RIGHT-TURN Queue Length (feet)
0
100
200
300
400
500
600
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.

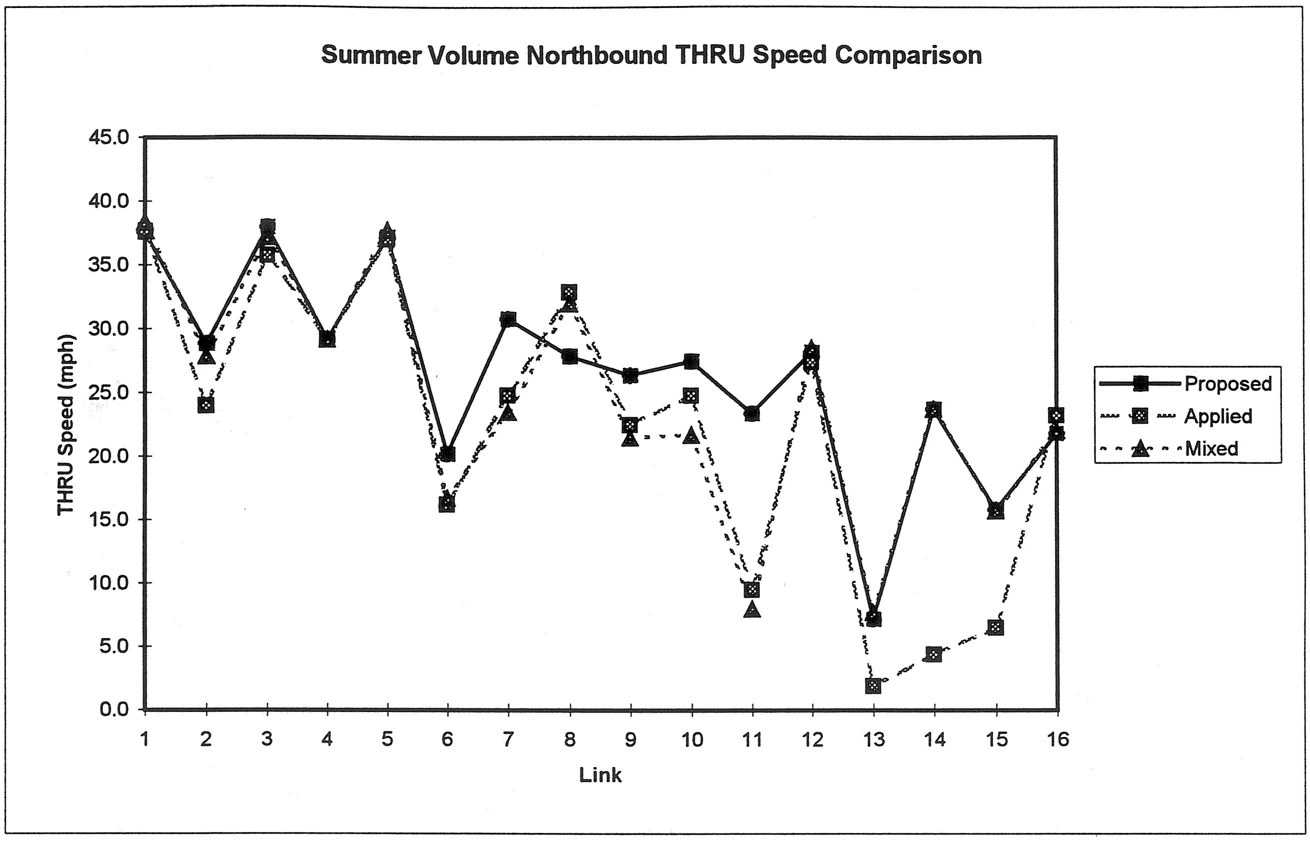

Summer Volume Southbound LEFT-TURN Queue Length Comparison
LEFT-TURN Queue Length (feet)
0
100
200
300
400
500
600
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.

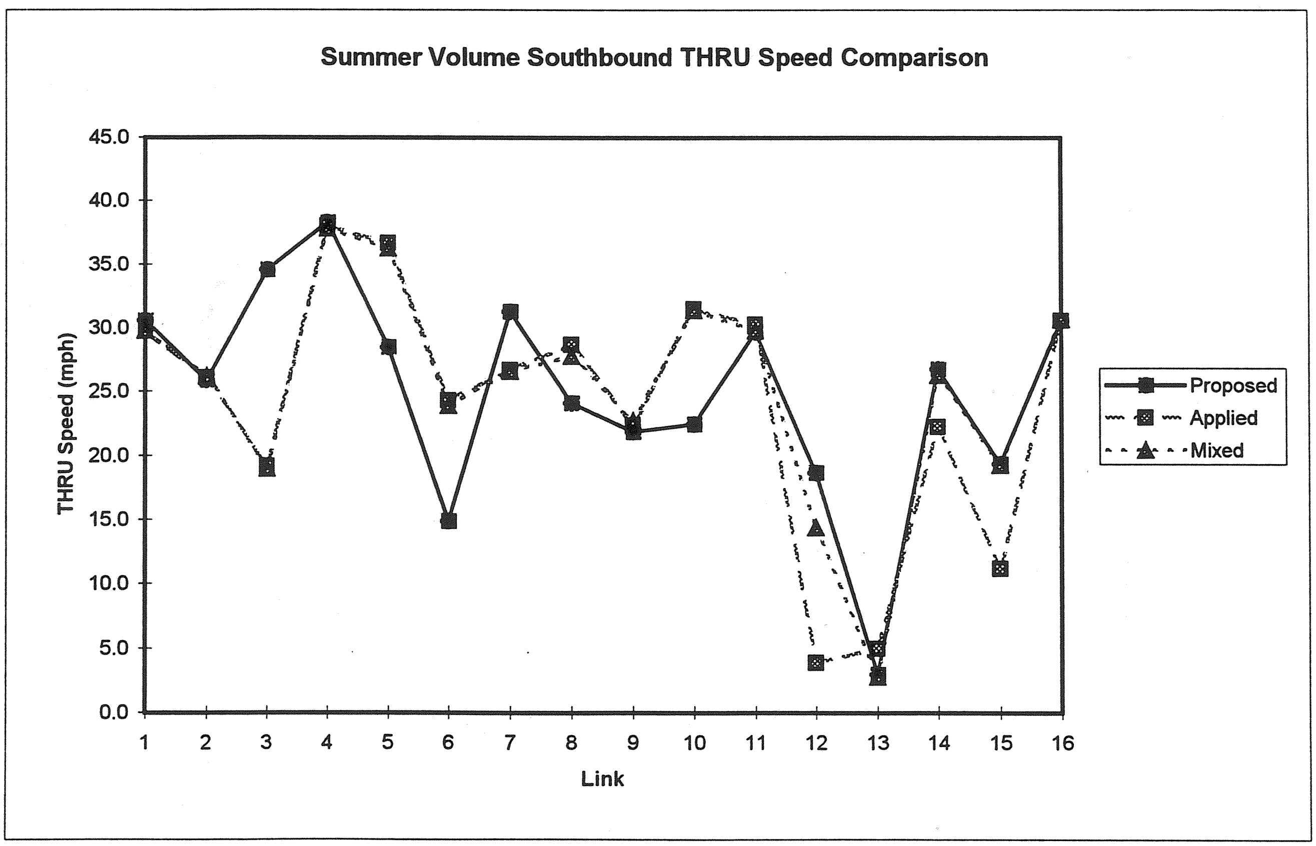

Summer Volume Northbound THRU Speed Comparison
THRU Speed (mph)
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed

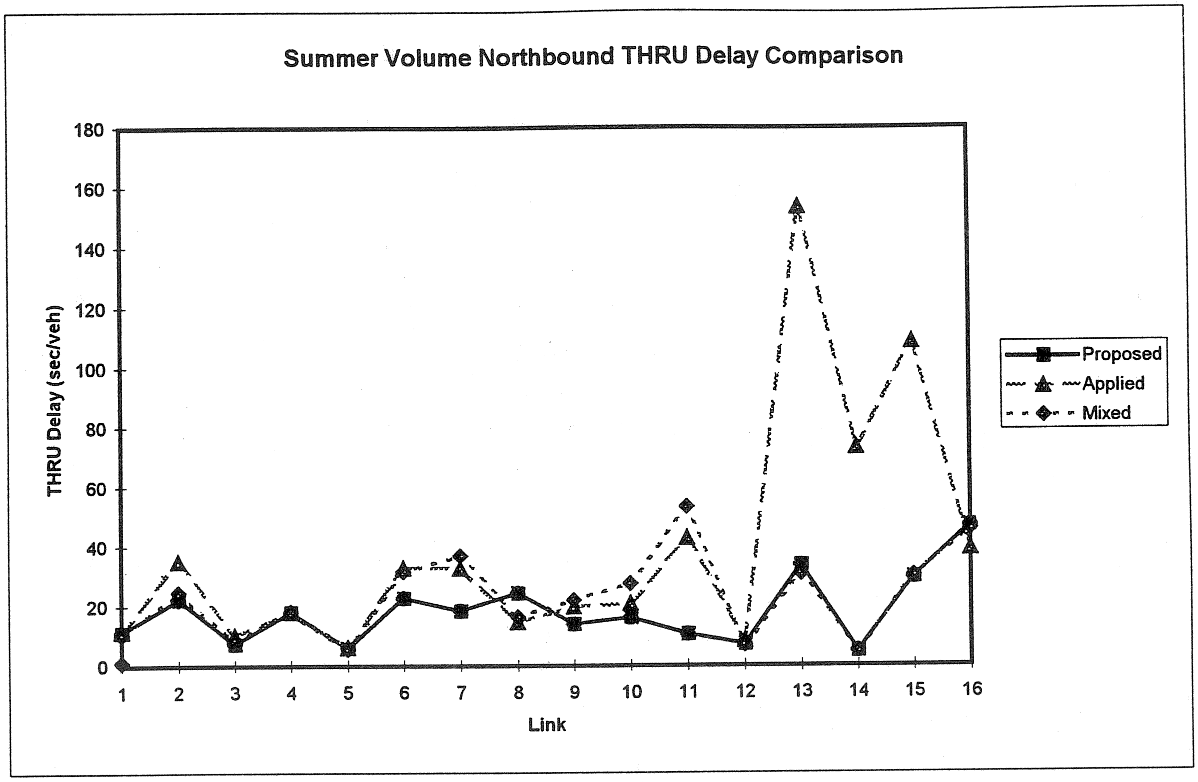

Summer Volume Southbound THRU Speed Comparison
THRU Speed (mph)
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed

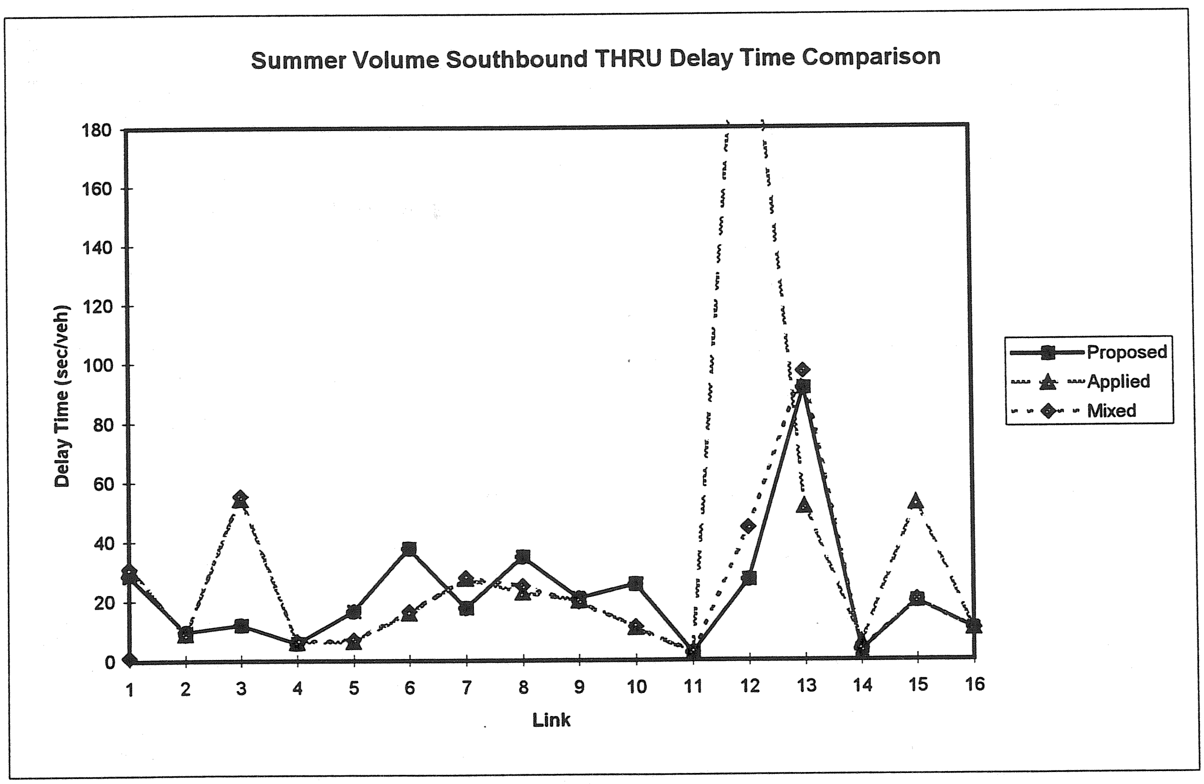

Summer Volume Northbound THRU Delay Comparison
THRU Delay (sec/veh)
180
160
140
120
100
80
60
40
20
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed

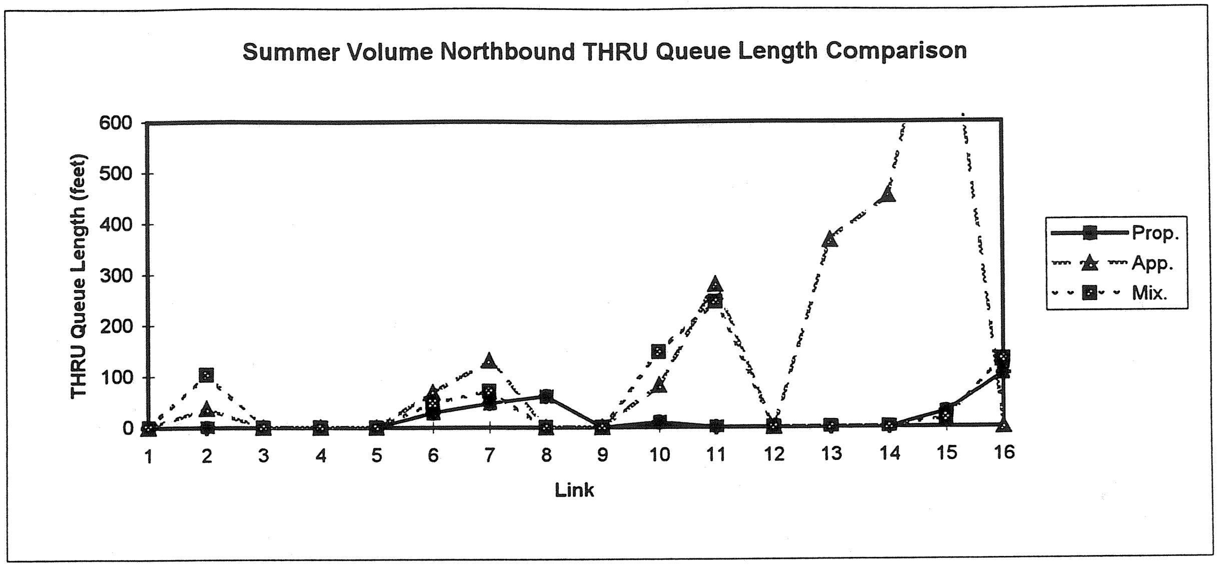

Summer Volume Southbound THRU Delay Time Comparison
Delay Time (sec/veh)
180
160
140
120
100
80
60
40
20
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed


Summer Volume Northbound THRU Speed Comparison
THRU Speed (mph)
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed

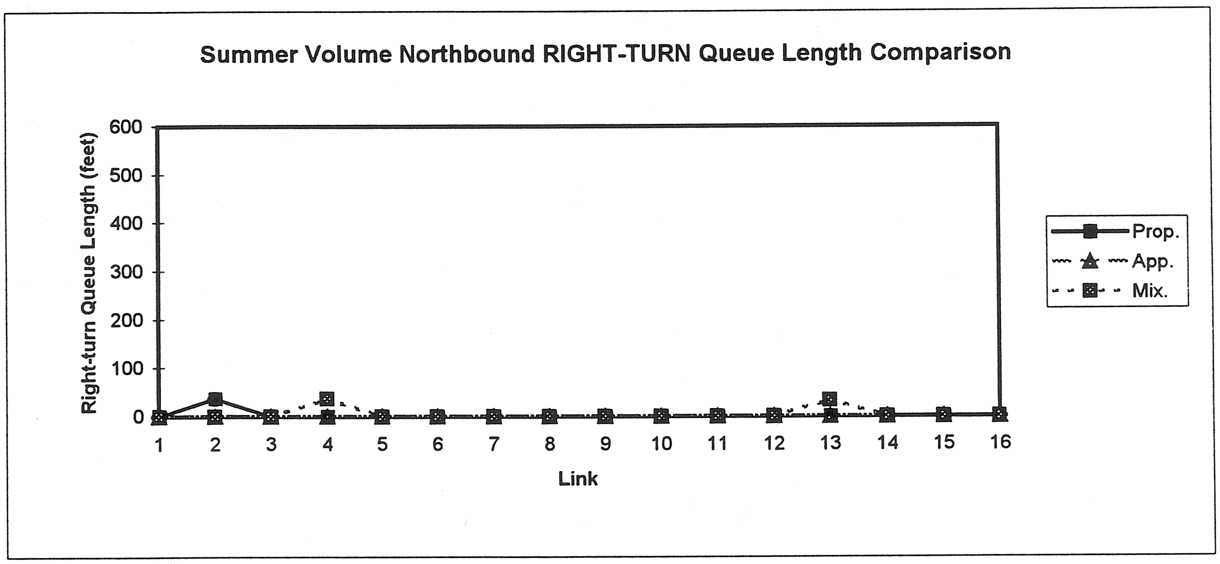

Summer Volume Southbound THRU Speed Comparison
THRU Speed (mph)
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed


Summer Volume Northbound THRU Delay Comparison
THRU Delay (sec/veh)
180
160
140
120
100
80
60
40
20
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed

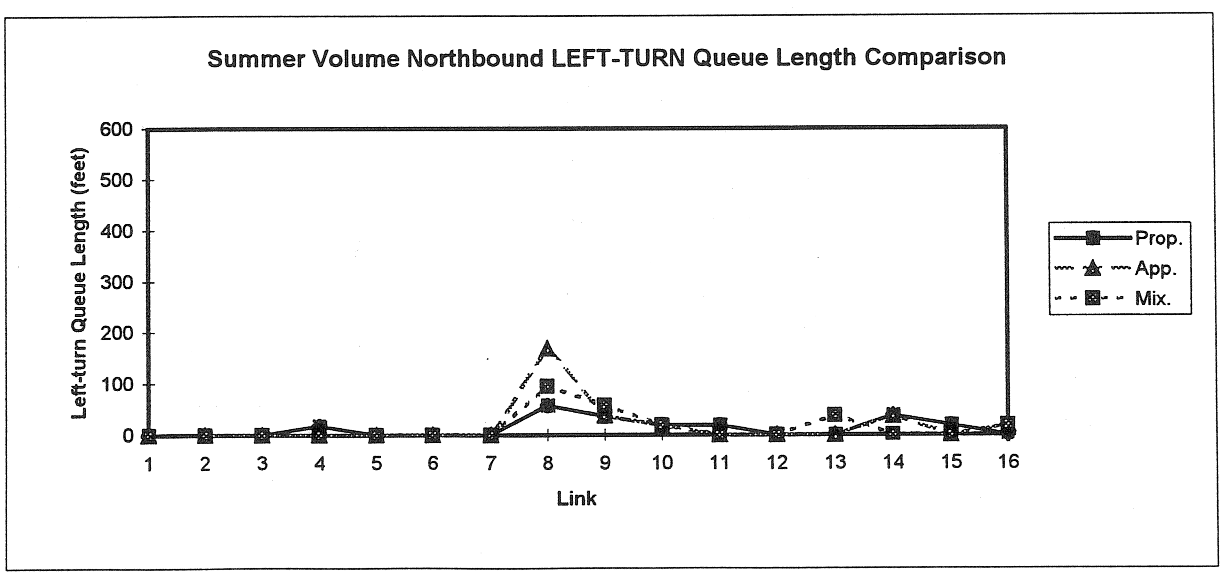

Summer Volume Southbound THRU Delay Time Comparison
Delay Time (sec/veh)
180
160
140
120
100
80
60
40
20
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed



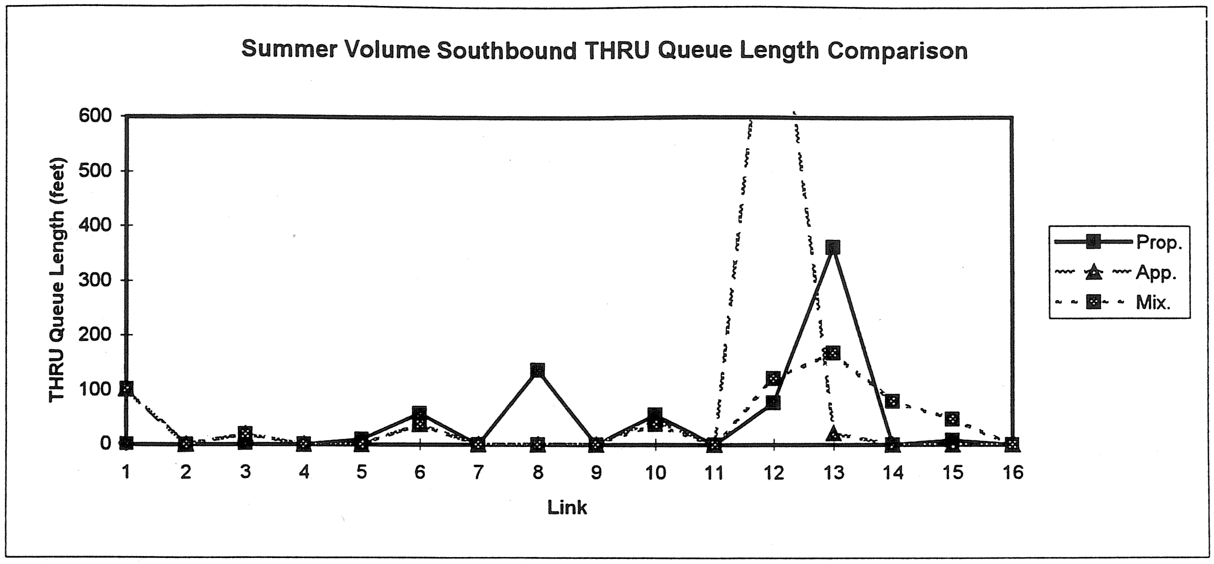
Summer Volume Northbound THRU Queue Length Comparison
THRU Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.

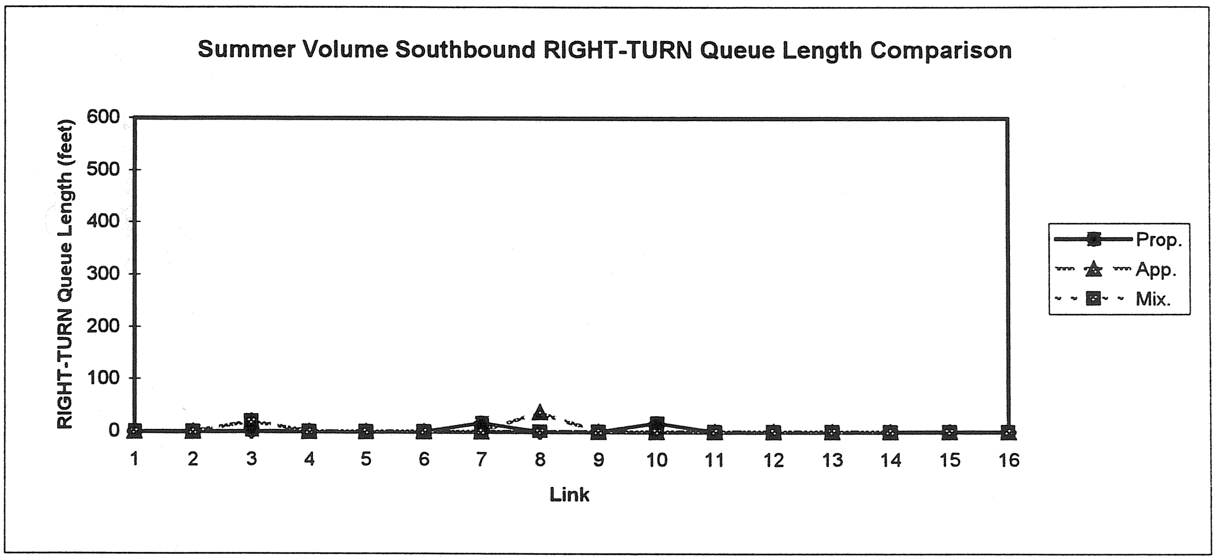


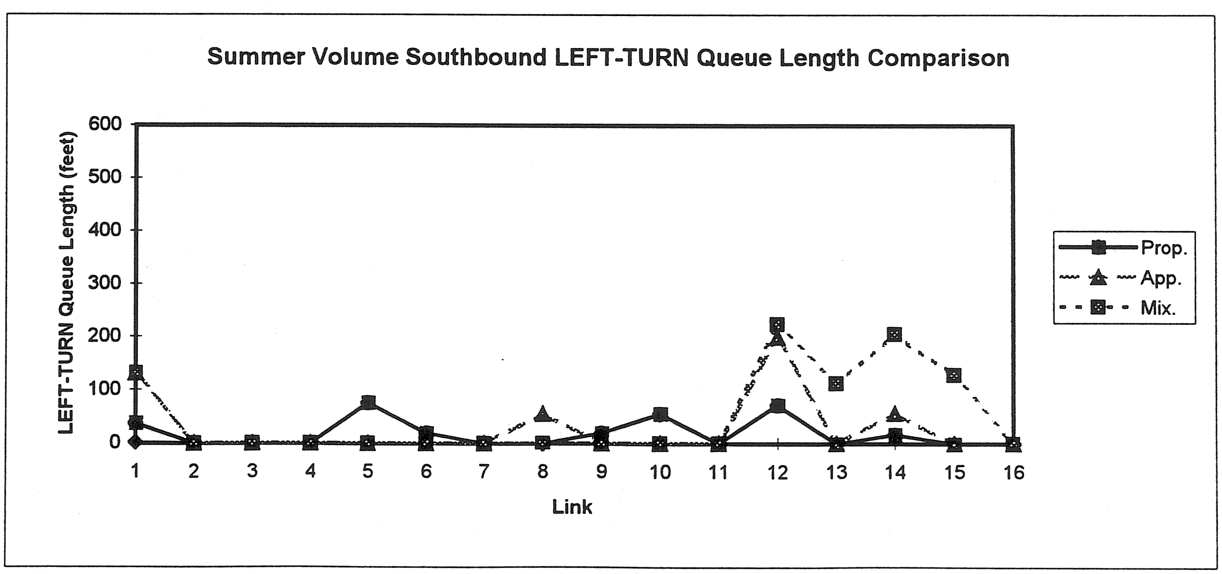
Summer Volume Northbound RIGHT-TURN Queue Length Comparison
Right-turn Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.



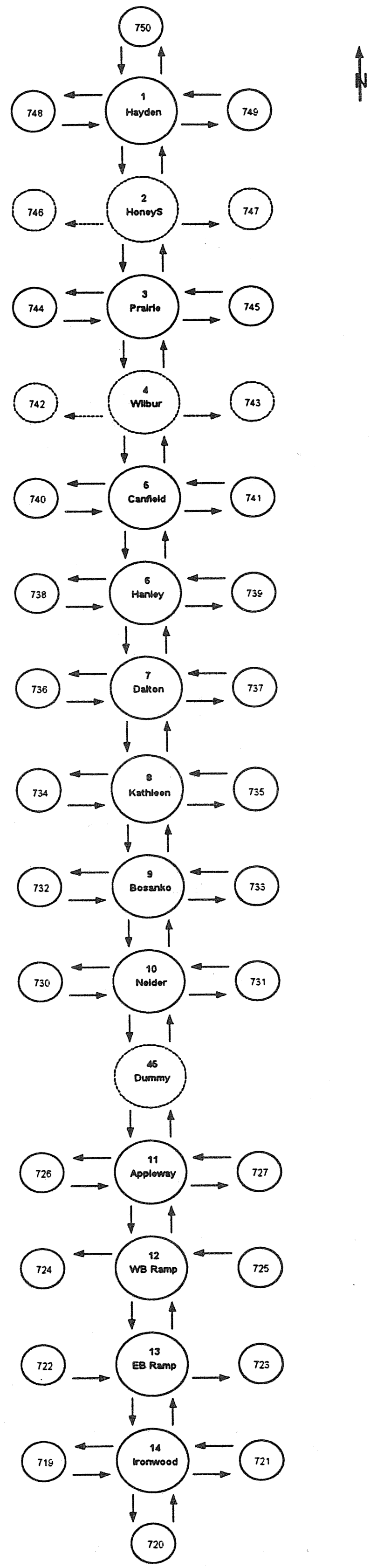
Summer Volume Northbound LEFT-TURN Queue Length Comparison
Left-turn Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.

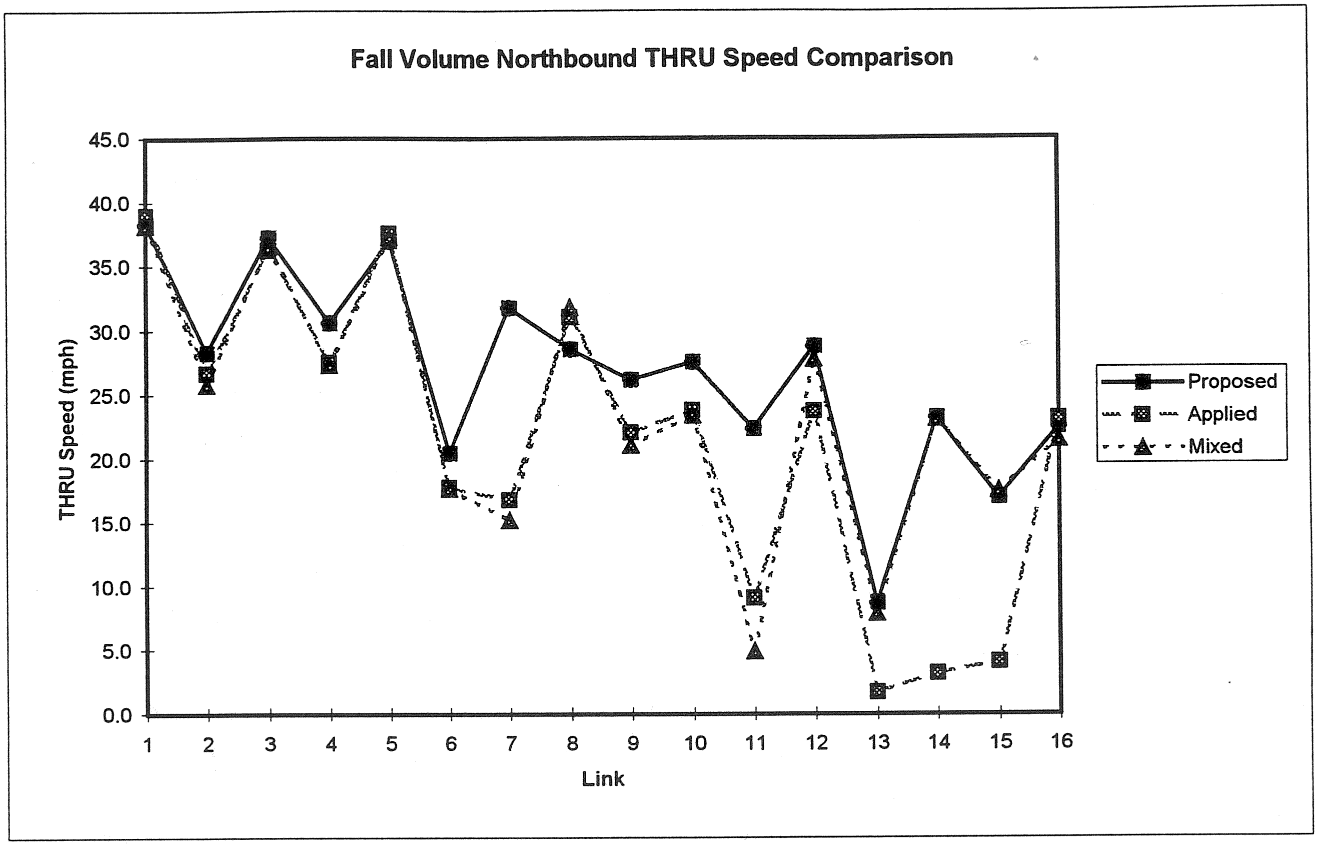

Summer Volume Southbound THRU Queue Length Comparison
THRU Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.


Summer Volume Southbound RIGHT-TURN Queue Length Comparison
RIGHT-TURN Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.

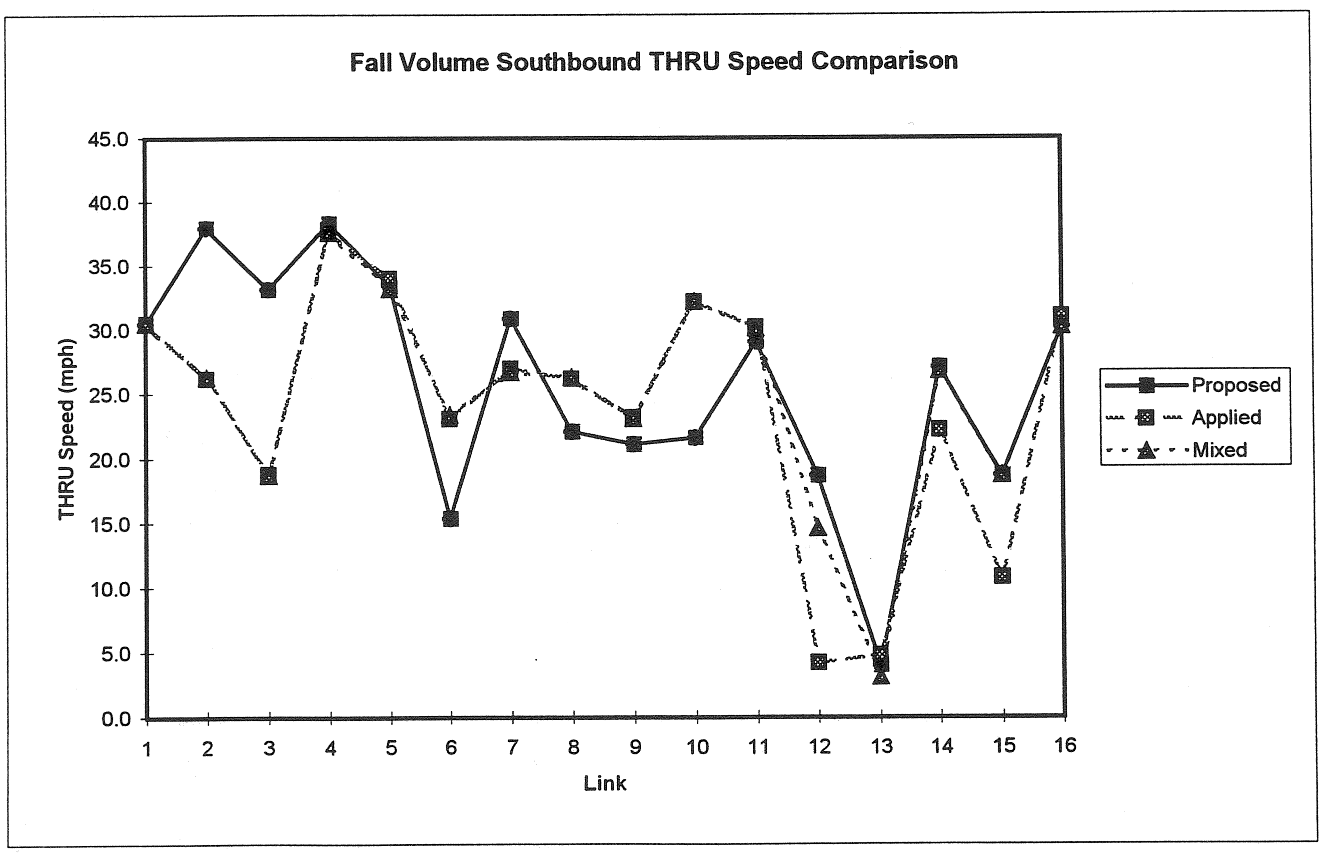

Summer Volume Southbound LEFT-TURN Queue Length Comparison
LEFT-TURN Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.

I-VII. PROPOSED, APPLIED, and MIXED Signal Timings Comparison under Fall Volume

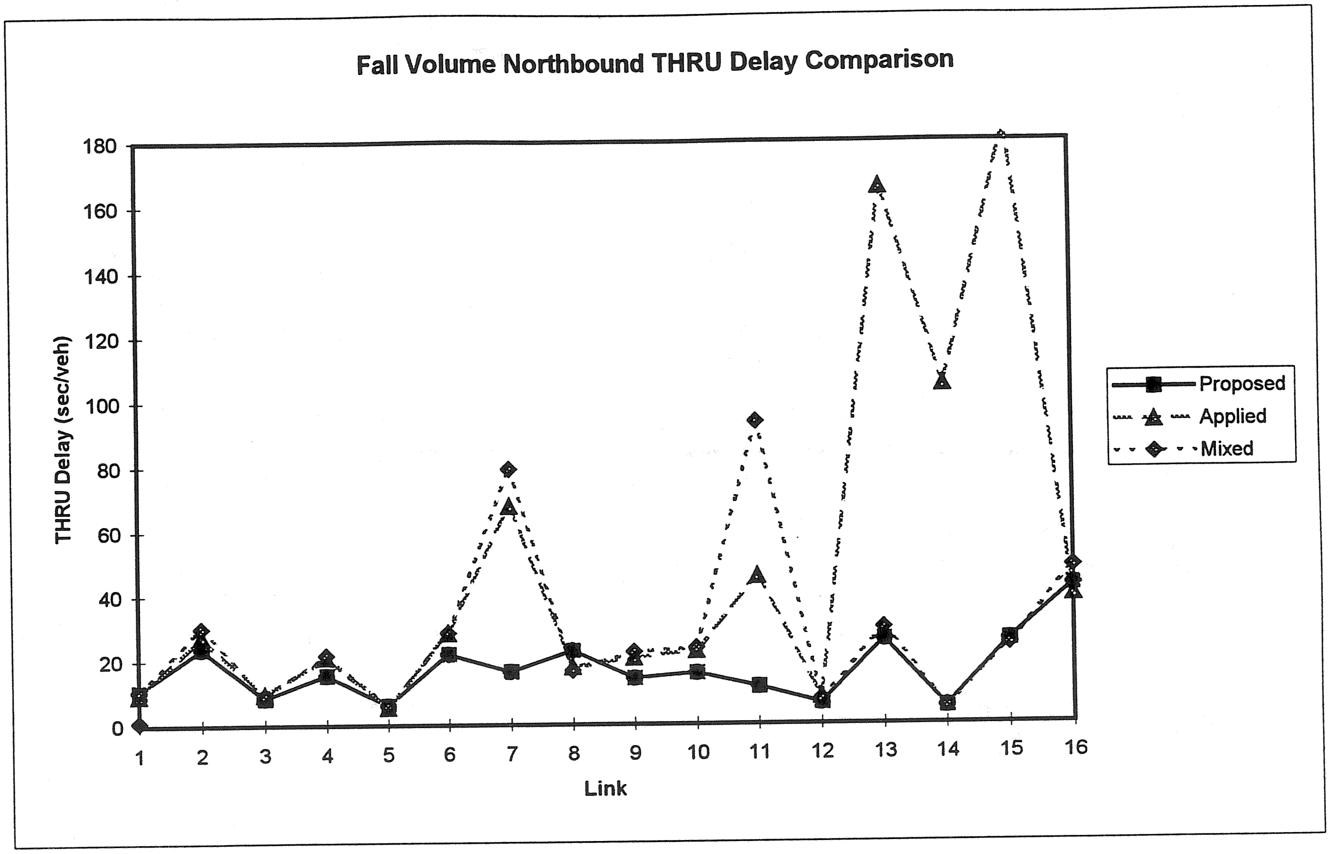

750
1
Hayden
748
749
2
HoneyS
746
747
3
Prairie
744
745
4
Wilbur
742
743
5
Canfield
740
741
6
Hanley
738
739
7
Dalton
736
737
8
Kathleen
734
735
9
Bosanko
732
733
10
Neider
730
731
45
Dummy
11
Appleway
726
727
12
WB Ramp
724
725
13
EB Ramp
722
723
14
Ironwood
719
721
720
N

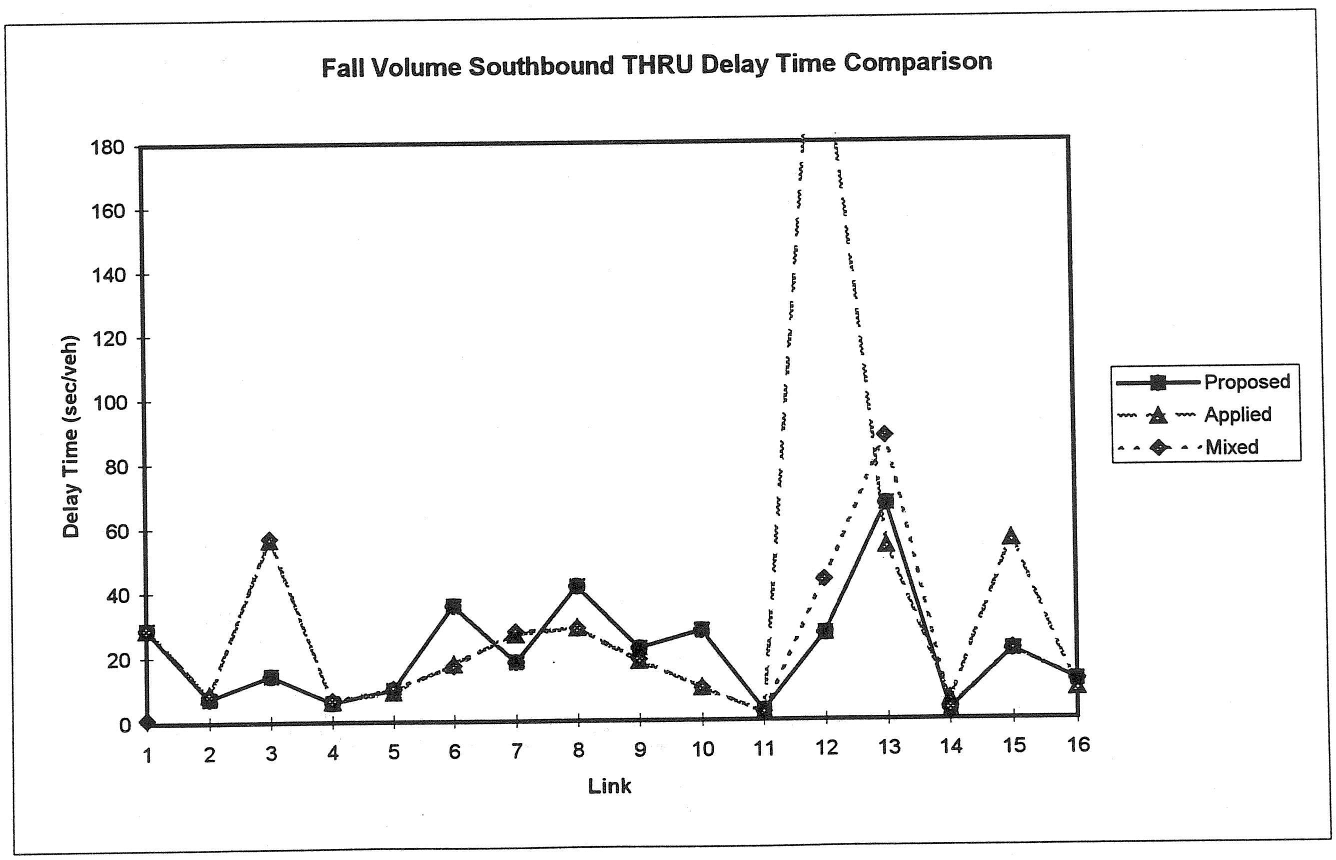


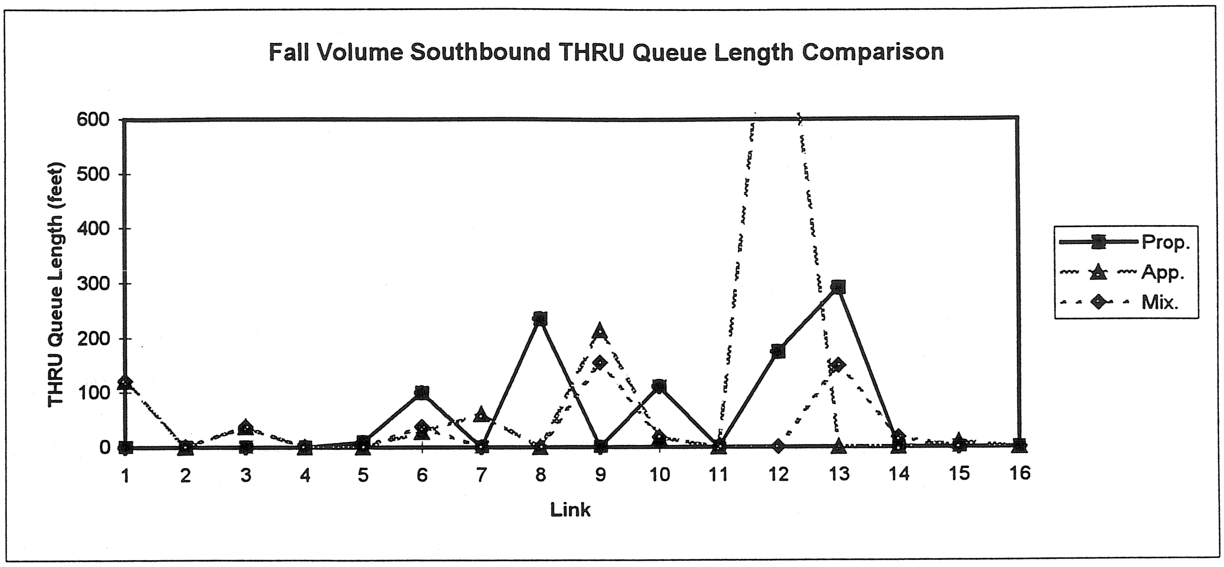
Fall Volume Northbound THRU Speed Comparison
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
THRU Speed (mph)
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed

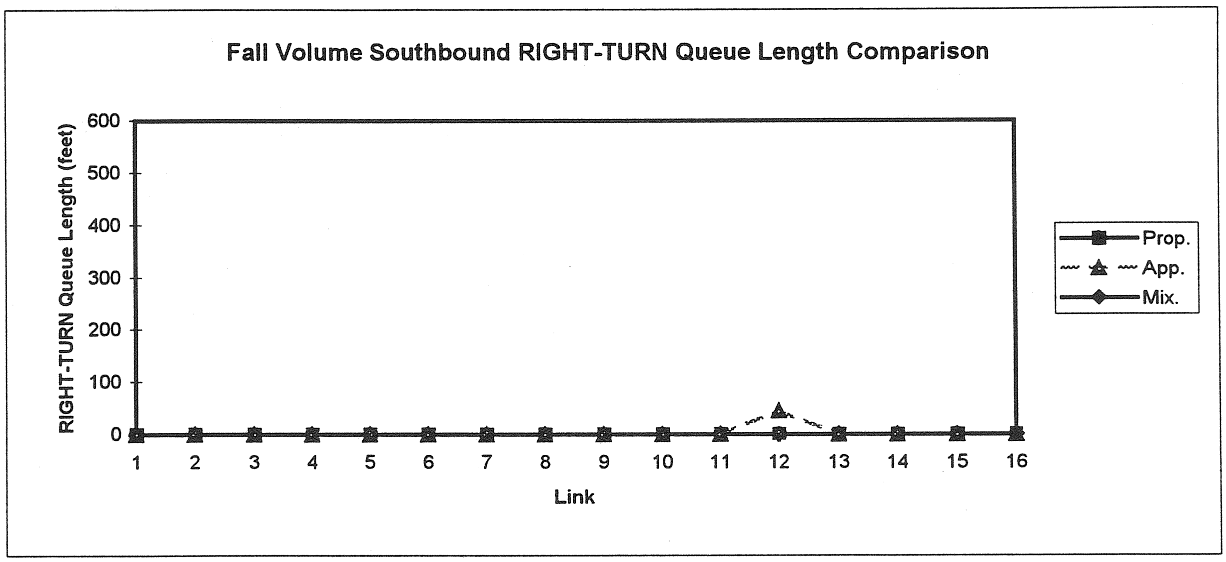


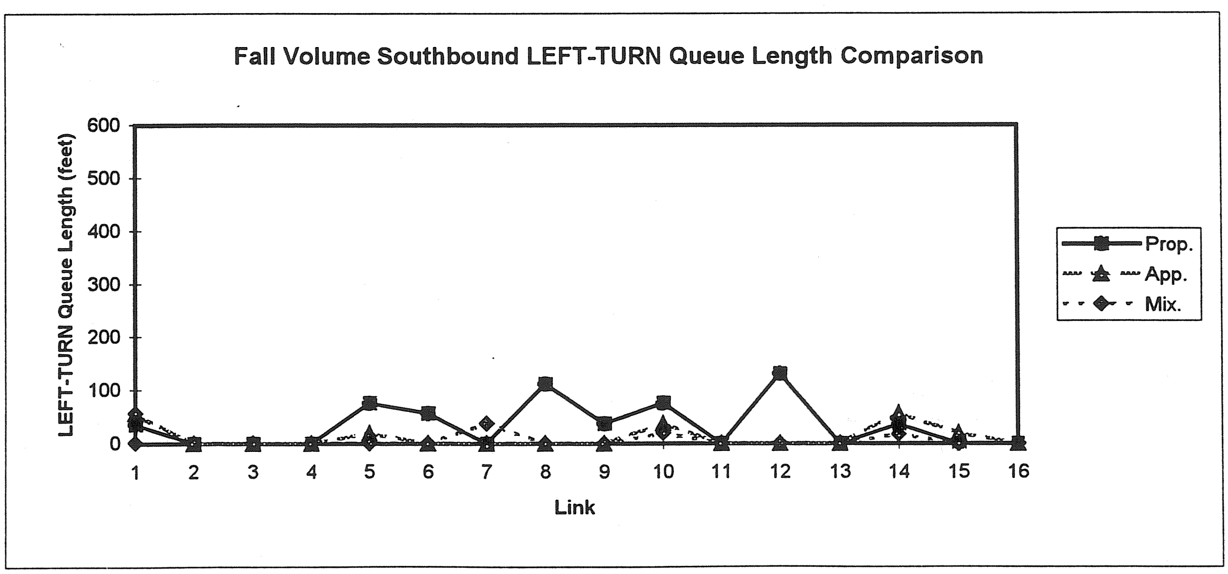
Fall Volume Southbound THRU Speed Comparison
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
THRU Speed (mph)
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed

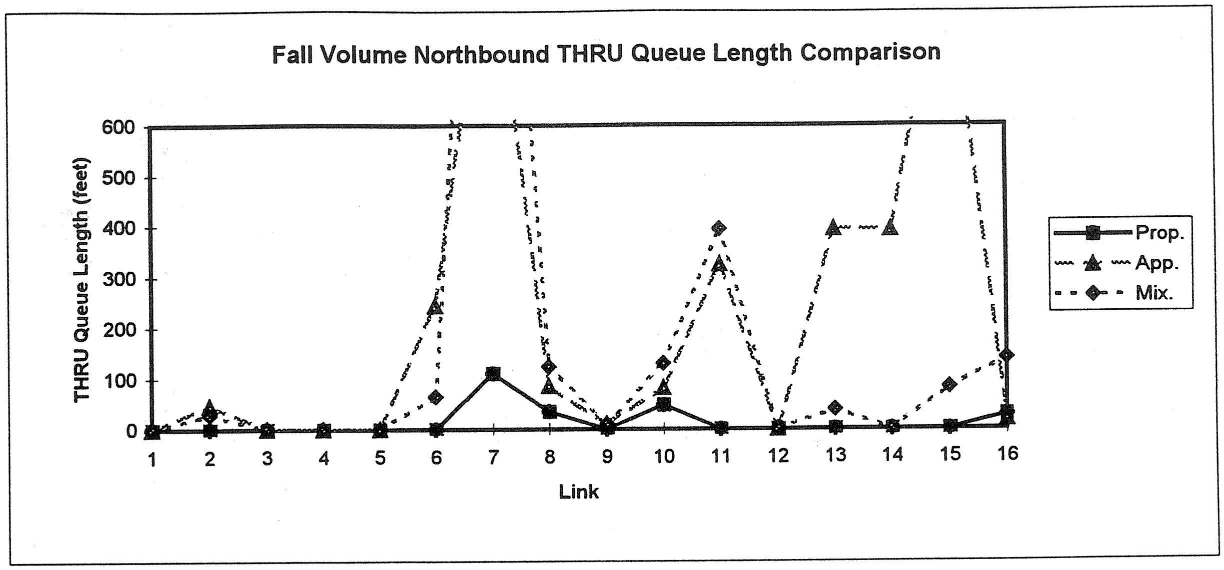

Fall Volume Northbound THRU Delay Comparison
THRU Delay (sec/veh)
0
20
40
60
80
100
120
140
160
180
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed

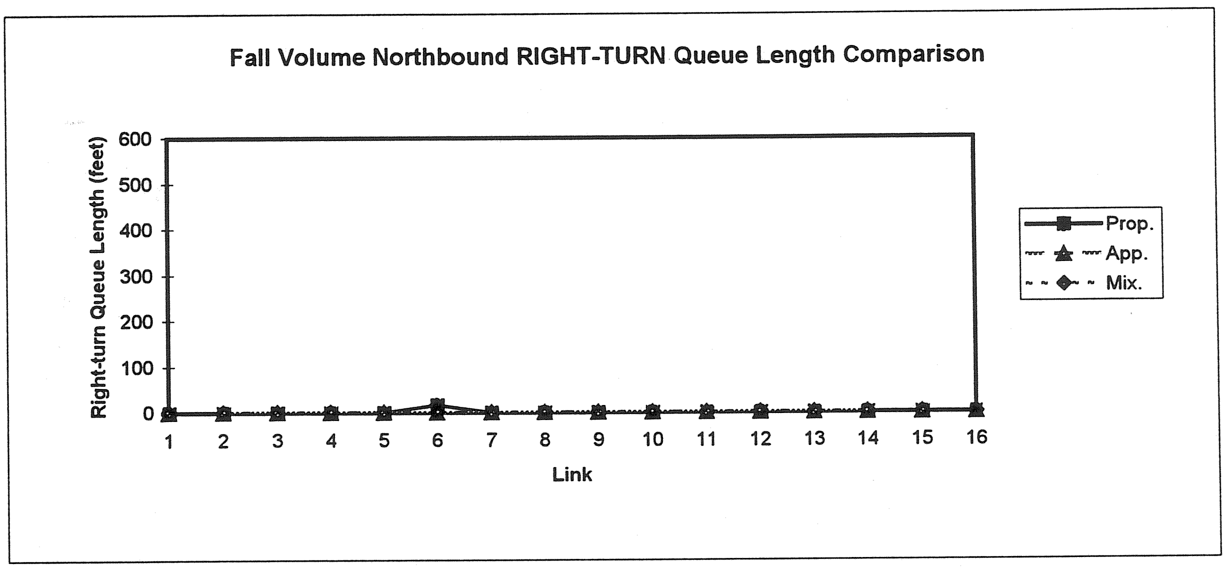

Fall Volume Southbound THRU Delay Time Comparison
Delay Time (sec/veh)
0
20
40
60
80
100
120
140
160
180
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Proposed
Applied
Mixed

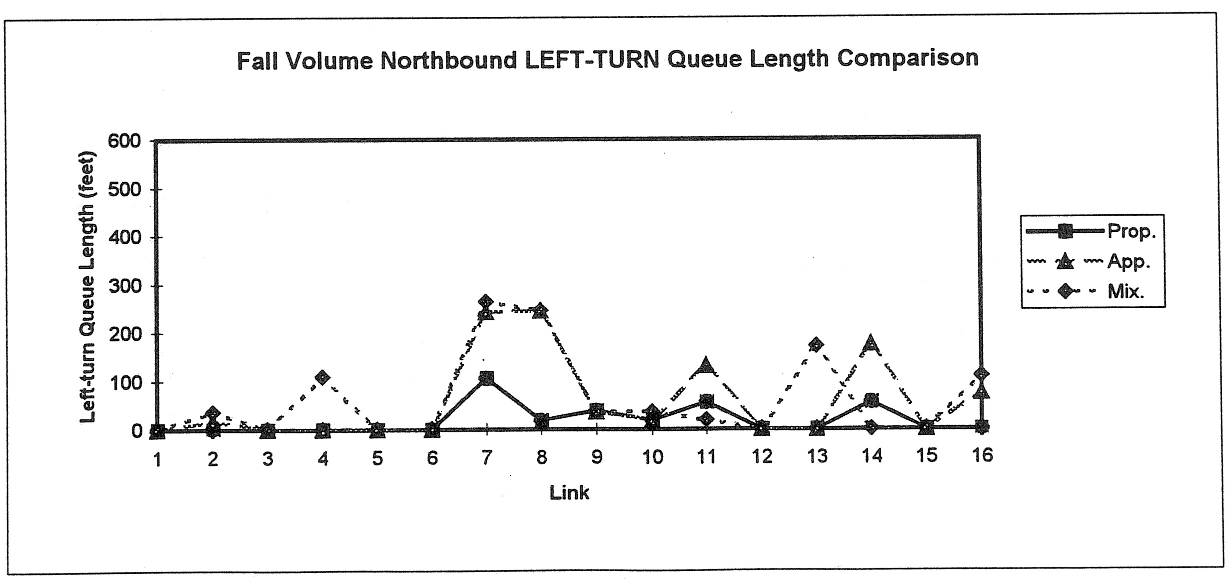

Fall Volume Southbound THRU Queue Length Comparison
THRU Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.


Fall Volume Southbound RIGHT-TURN Queue Length Comparison
RIGHT-TURN Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.

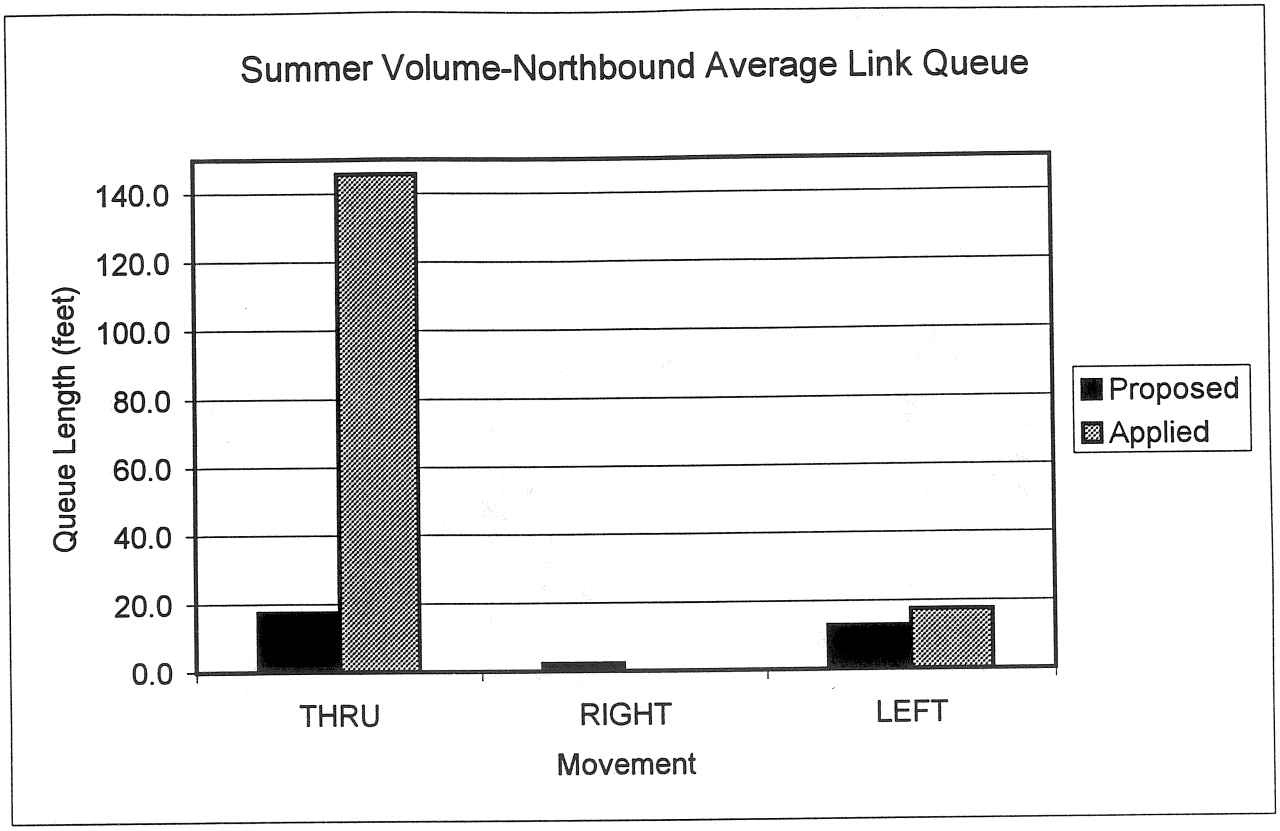

Fall Volume Southbound LEFT-TURN Queue Length Comparison
LEFT-TURN Queue Length (feet)
600
500
400
300
200
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Link
Prop.
App.
Mix.

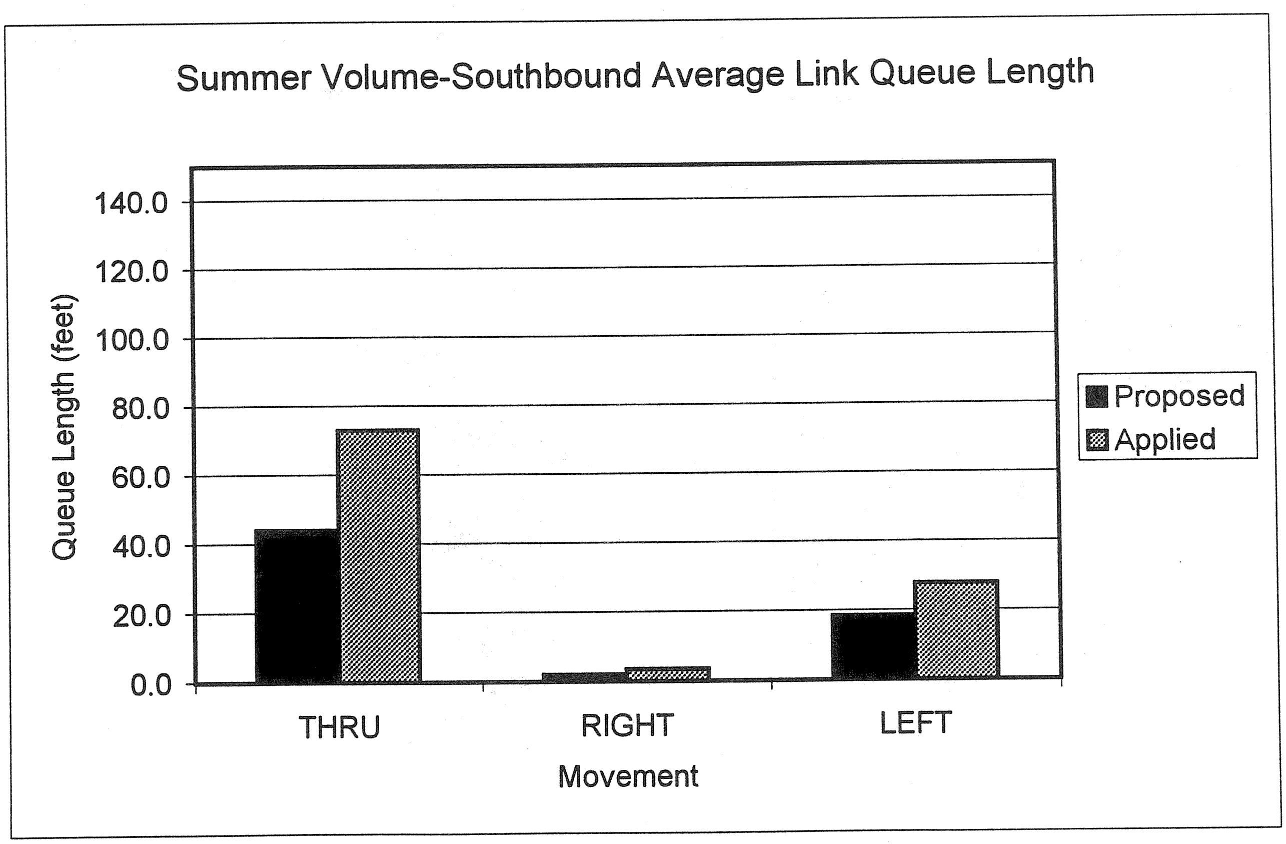

Fall Volume Northbound THRU Queue Length Comparison
THRU Queue Length (feet)
600
500
400
300
200
100
0
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16
Link
Prop.
App.
Mix.

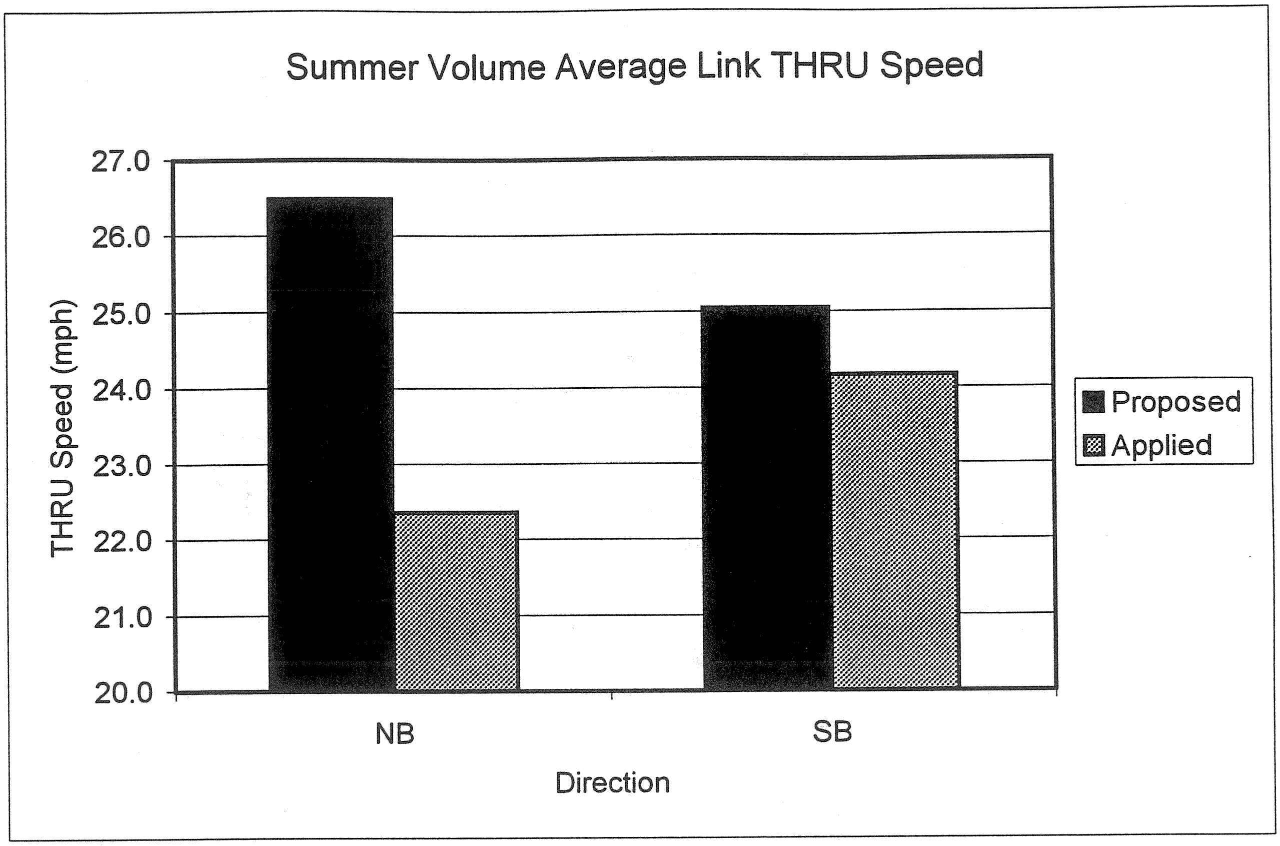

Fall Volume Northbound RIGHT-TURN Queue Length Comparison
Right-turn Queue Length (feet)
600
500
400
300
200
100
0
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16
Link
Prop.
App.
Mix.

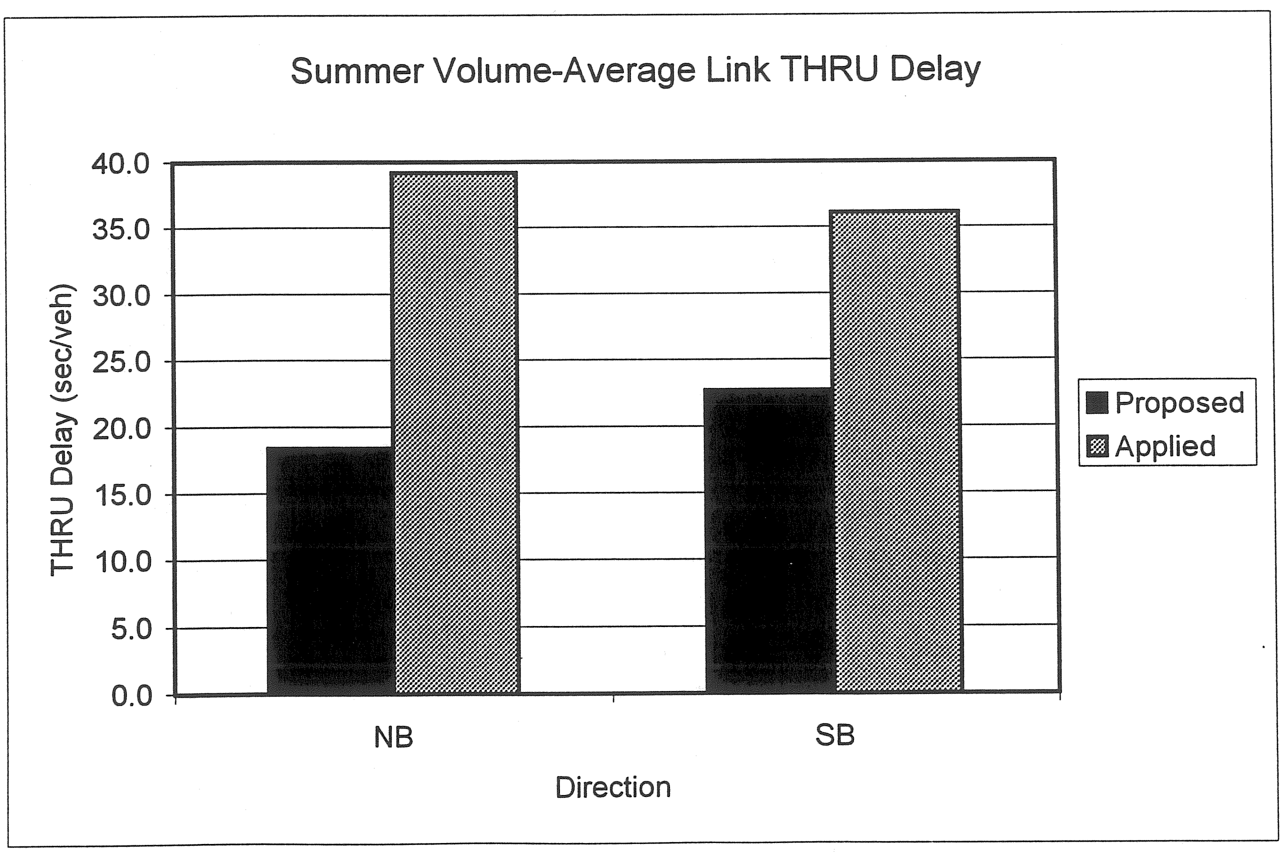

Fall Volume Northbound LEFT-TURN Queue Length Comparison
Left-turn Queue Length (feet)
600
500
400
300
200
100
0
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16
Link
Prop.
App.
Mix.

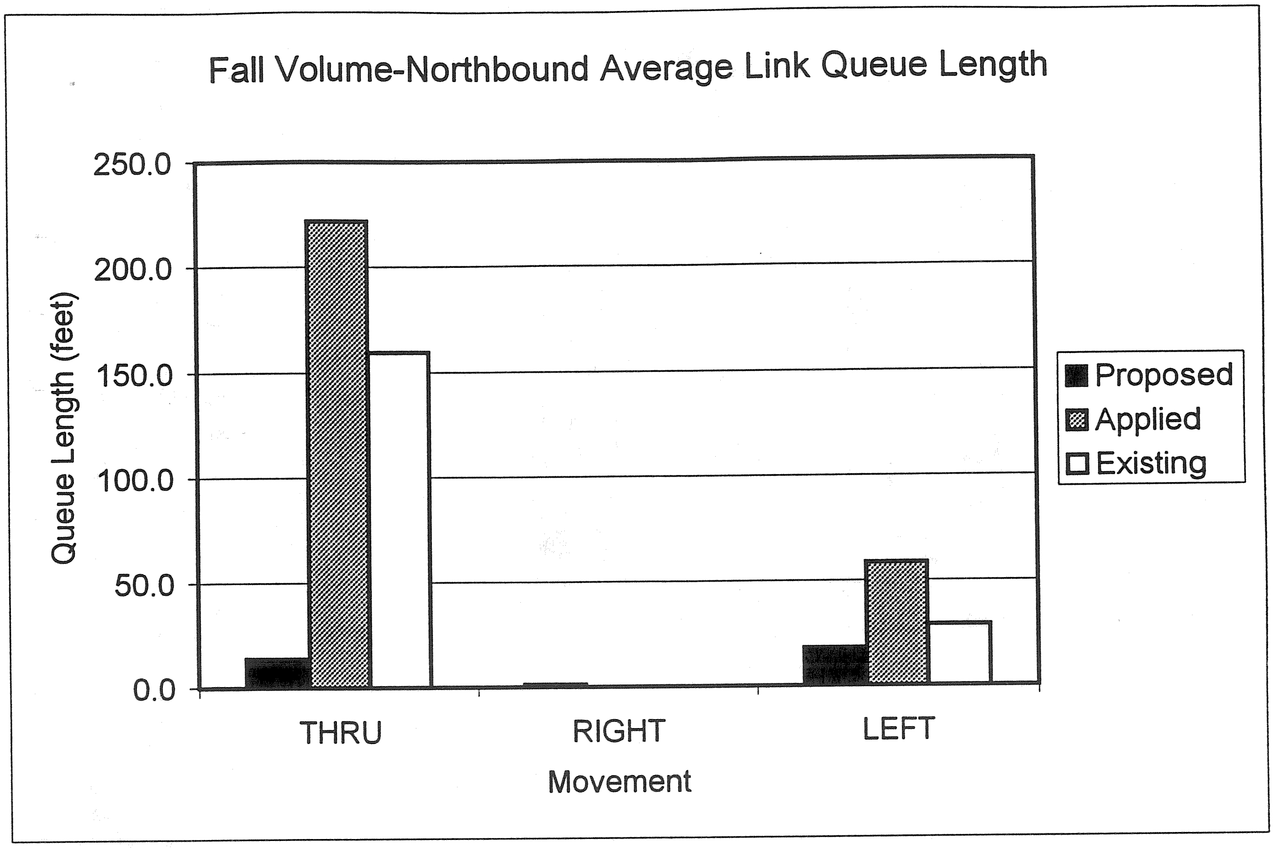
I-VIII Overall Arterial MOEs Comparison

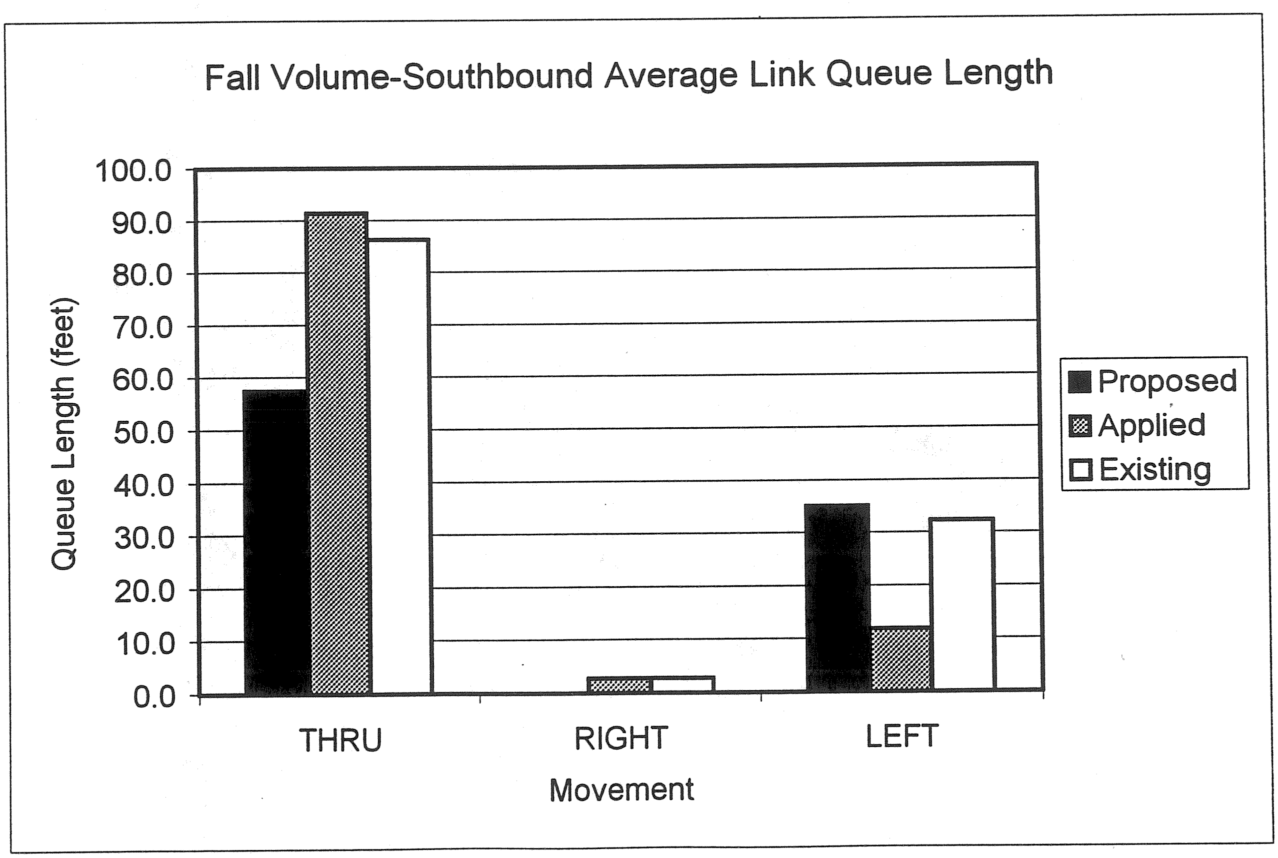

Summer Volume-Northbound Average Link Queue
Queue Length (feet)
140.0
120.0
100.0
80.0
60.0
40.0
20.0
0.0
THRU
RIGHT
LEFT
Movement
Proposed
Applied

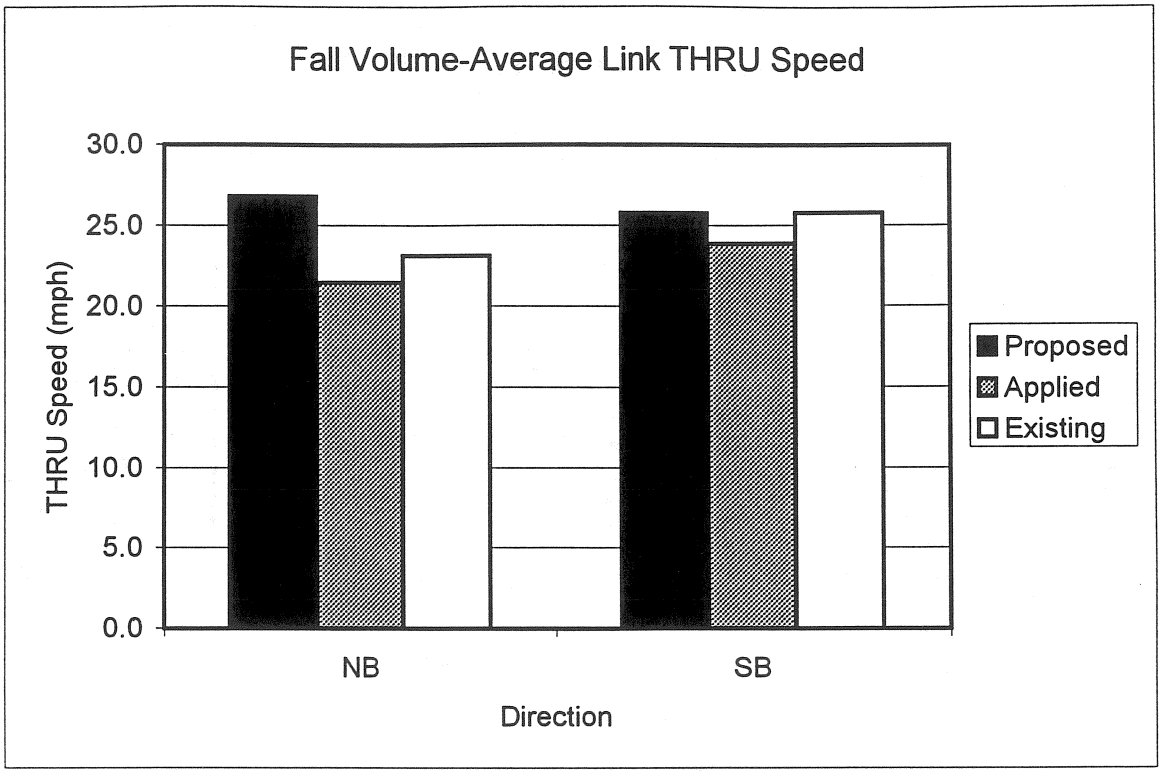

Summer Volume-Southbound Average Link Queue Length
Queue Length (feet)
140.0
120.0
100.0
80.0
60.0
40.0
20.0
0.0
THRU
RIGHT
LEFT
Movement
Proposed
Applied

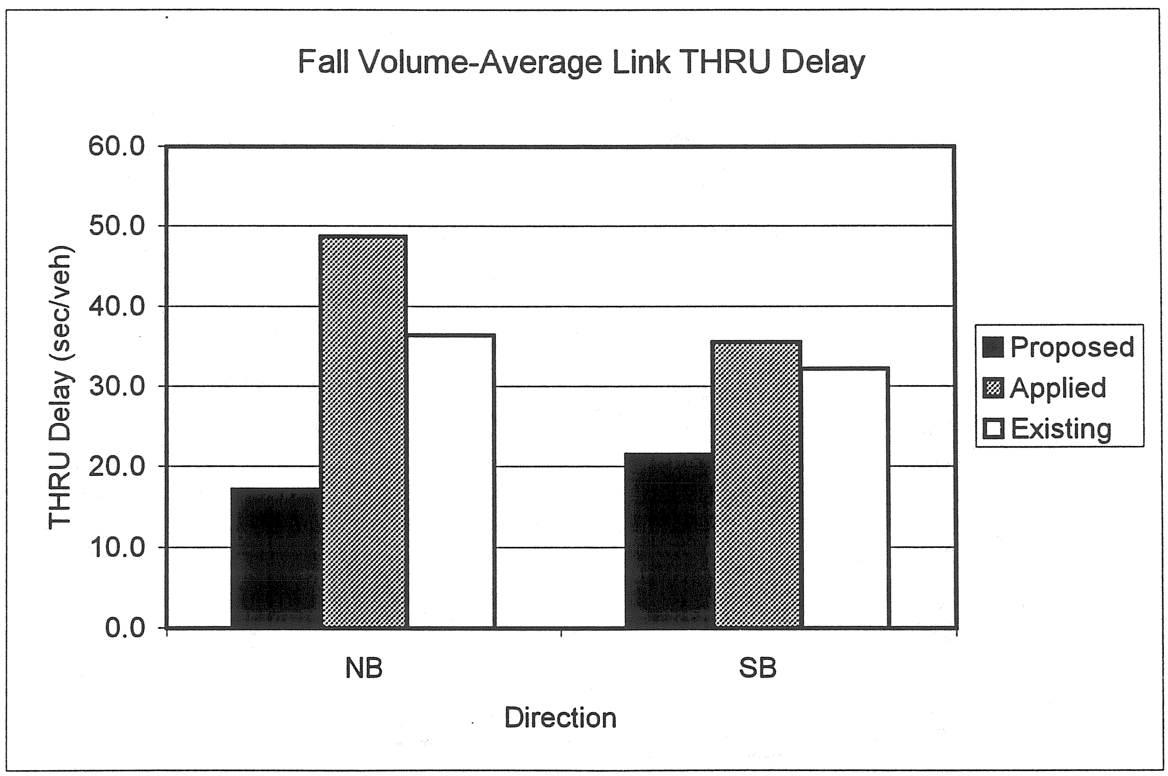

Summer Volume Average Link THRU Speed
27.0
26.0
25.0
24.0
23.0
22.0
21.0
20.0
THRU Speed (mph)
NB
SB
Direction
Proposed
Applied


Summer Volume-Average Link THRU Delay
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
THRU Delay (sec/veh)
NB
SB
Direction
Proposed
Applied


Fall Volume-Northbound Average Link Queue Length
250.0
200.0
150.0
100.0
50.0
0.0
Queue Length (feet)
THRU
RIGHT
LEFT
Movement
Proposed
Applied
Existing


Fall Volume-Southbound Average Link Queue Length
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
Queue Length (feet)
THRU
RIGHT
LEFT
Movement
Proposed
Applied
Existing


Fall Volume-Average Link THRU Speed
THRU Speed (mph)
30.0
25.0
20.0
15.0
10.0
5.0
0.0
NB
SB
Direction
Proposed
Applied
Existing


Fall Volume-Average Link THRU Delay
THRU Delay (sec/veh)
60.0
50.0
40.0
30.0
20.0
10.0
0.0
NB
SB
Direction
Proposed
Applied
Existing